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BRIDGE DEPARTMENT

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State of California
Business and Transportation Agency
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Bridge Department

OVERHEAD BOX BEAM SIGN

Report Prepared in Design Section 12 of the

Bridge Department by Thomas L. Pollock

624125

HPR-PR-1(7) DO 460

J. E. McMahon, Assistant State Highway Engineer - Bridges

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In Cooperation with

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ABSTRACT

Reference: Pollock, T. L., "Overhead Box Beam Sign," State of California, Business and Transportation Agency, Department of Public Works, Division of Highways, Bridge Department. Research and Development No. 7-69, December, 1969.

Abstract: The results of full scale tests on cantilevered box beam signs are analyzed. A total of four tests were made, two each on an aluminum structure and a steel structure. The structures were of the "balanced butterfly" type, but the cantilevered arms were tested separately. One arm used plug-weld web connections and the other used huckbolt fasteners.

Of particular concern was the response of the 16 gage ribbed sheet metal web panels to direct shear.

Test measurements are compared with theoretical values wherever possible and some design criteria are recommended.

Key Words: Aluminum, box beam signs, chord angles, deflections, huck-bolts, load factors, plug welds, puddle welds, ribbed sheet metal webs, steel, torsion, web shear strength.

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ACKNOWLEDGMENT

This project was performed in cooperation with the Materials and Research Department of the California Division of Highways and the U. S. Department of Transportation, Federal Highway Administration, Bureau of Public Roads.

The author wishes to expressly thank Messrs. Wallace Ames, Robert Doty, and Vincent Martin for conducting all testing and their assistance in interpreting the test data.

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TABLE OF CONTENTS

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LIST OF FIGURES

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I. INTRODUCTION

The current emphasis on aesthetics in highway design has directed attention to the desirability of an alternative to the open truss sign frames that have been used along California freeways for many years. Approximately four years ago Mr. R. C. Cassano, Supervising Bridge Engineer of the Bridge Department, decided to adapt ribbed sheet metal panels to sign design based on tests run on this material at Cornell University. The first sign using 16 ga. ribbed sheet metal was designed at that time and immediately won approval for its increased aesthetic qualities from both the Traffic and Bridge Departments of the California Division of Highways. The first sign had two side webs 2'-0" apart with solid top and bottom flanges giving it a box like appearance, thus the name "Box Beam" sign.

The new design, utilizing webs of vertically ribbed sheet metal connected to the flange members by puddle welds, was subsequently incorporated into contract plans by the Division of Highways Bridge Department. The name "Box Beam" was retained even though the flanges were changed from solid members to two chord angles joined by small wind bracing angles. However, these design features had not previously been used in this manner and no information regarding the load carrying capacity of ribbed sheet metal used in this application was available. In an effort to

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substantiate and further develop this design with due regard for economy and safety, the Bridge Department initiated a research project. It consisted of fabricating and testing to failure two full size sign structures, one of steel and one of aluminum, using puddle welds and Huckbolt fasteners for web to chord-angle connections.

Physical testing was selected as the best method of providing information regarding the following:

1. The ultimate shear capacity of 16 ga. ribbed sheet metal webs in this application.
2. The effectiveness of web to chord-angle connections using puddle welds or huckbolts.
3. The total load capacity of the aluminum structure and the steel structure.
4. Deflections of the cantilevered arms.

This report will analyze the test data and relate it to design assumptions. The strength of various components of the structural system will be considered. Suggestions for design criteria including factors of safety will be advanced and extrapolation of test results discussed.

Of principle concern is the use of test data for future designs, applications as well as limitations.

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II. BOX BEAM SIGN

The box beam sign is essentially two parallel "girders" connected at the top & bottom by light bracing to ensure the distribution of wind loads. The "flanges" are chord angles and the webs are light gage ribbed sheet metal.

For design purposes the chord angles are assumed to carry the entire bending moment and the ribbed sheet metal webs are assumed to exclusively carry the shear. The web to chord-angle connections must be of sufficient strength to ensure the effectiveness and integrity of the girder.

A complete description of the box beam structure is given in Appendix "A" in the section entitled "The Sign Structures."

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III. ANALYSIS OF TEST DATA

CHORD-ANGLES

The cross-section of the cantilever beam is shown by Section A-A of the Box Beam Sign Structure layout and Details Sheet (Figure 8). For design it is considered sufficiently accurate to assume that the chord angles are the sole moment carrying members and are stressed uniformly throughout their cross-section. Thus moment stress is equal to:

$\frac{\text{Moment}}{4AY}$, where A = Area of chord angle

Y = Distance from the chord angle center of gravity to the ℄ of the box section.

The moment of Inertia, I, is assumed equal to $4AY^2$.

Figures 1 through 5 indicate a fairly good correlation between calculated and measured stress in the chord angles. An exception is the lower chord-angles of the Huck-bolted steel sign (not shown on graphs) in which gage readings indicated considerably less than calculated stress. Since there was no apparent reason for these reduced readings they were assumed erroneous.

In general, the test data supports the design assumptions and there appears to be little buckling tendency in the compression chord-angles. At maximum loading the steel chord-angles were stressed to approximately 25.2 ksi

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(18.3 ksi allowable)[1] and the aluminum chord-angles were stressed to approximately 11.8 ksi (10 ksi allowable)[2].

RIBBED SHEET METAL WEB

Most of the design criteria used for shear panels of light gage steel are based upon empirical tests. The design of box beam sign webs in current use in California is based upon an empirical study by Arthur H. Nilson at Cornell University.[3] Although some recent publications[4,5] have suggested an analytical approach, the complexity of the problem requires the use of test data for design.

Previous tests have indicated that shear strength of a ribbed web is dependent upon many variables including:

1. Depth of web.
2. Thickness of web.
3. Geometrical configuration of web.
4. Modulus of Elasticity.
5. Type of connection to main chords.
6. Panel splice detail.
7. Material

1 $20{,}000 - 7.5\frac{(360)^2}{(24)} = 18.3$ psi.

2 Within 1" of weld @ point of lateral support.

3 Arthur H. Nilsen, "Shear Diaphragms of Light Gage Steel", _Journal of the Structural Division_, Vol. 86 No. St 11, Proceedings of the ASCE, November 1960.

4 John T. Easley & David E. McFarland, "Buckling of Light Gage Corrugated Metal Shear Diaphragms", _Journal of the Structural Division_, Vol. 95 No. St 7, Proceedings of ASCE, July 1969.

5 Erin R. Bryan & Wagih M. El-Dakhakhni, "Shear Flexibility and Strength of Corrugated Decks," _Journal of the Structural Division_, Vol. 94 No. St 11, Proceedings of ASCE, November 1968.

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The tests at Cornell[1] indicated that a single web of similar configuration was less than half as strong in shear as the double web of this test. This difference could be the result of fastening the chord-angle to the opposite side of the web or the possibility that the double web has more than twice the buckling strength of the single web.

In order to facilitate the use of test data, web shear resistance is depicted as a shear capacity per foot of web. This test indicated the following values at failure[2]:

Sign Structure	Web-Flange Connection	Shear Load on Web
Steel	Plug weld[3]	5,380 lbs/ft
Steel	Huckbolt	5,800 lbs/ft
Aluminum	Plug weld	2,720 lbs/ft
Aluminum	Huckbolt	2,750 lbs/ft

If the shear strength of the aluminum web is assumed proportional to the modulus of elasticity and the web thickness squared[4], the predicted aluminum shear strength would be:

$$5{,}800 \frac{(.0630)^2}{(.0598)} \frac{10}{29} = 2{,}200 \text{ lbs/ft}$$ compared to actual failure of 2,750 lbs/ft. This difference might be due to a

1 Nilsen, p. 124.

2 The implied assumption is that the web carries the entire shear. This is probably approximately correct, but the strength of the frame without panels was not tested.

3 This test terminated prior to decisive failure. See Appendix A.

4 Cornell tests showed good correlation between failure load and thickness squared. Nilsen, p. 124.

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different "buckling coefficient". Further testing would be required to support this supposition.

Current AASHO practice uses shear divided by cross-sectional area for determining the design shear stress in a flat web plate. Although this criterion is not satisfactory for ribbed webs, figure 6, shows rather good correlation between calculated (V/A) and measured shear stress for the structure tested.

Load Factors. Safety factors must be assigned to ultimate loads to insure against structural failure. Winter[1] states that load factors in current use range from 2.0 to 2.3 for dead load and 2.1 to 2.5 for wind loads. Nilsen[2] suggests a safety factor of 4 or an "effective safety factor" of 3 after the allowable increases to basic stresses are applied. The AASHO specification allows a 25 percent overstress for dead load. Wind loads do not produce shear stresses in the box beam sign. Therefore a safety factor of 4 produces an effective safety factor of $4/1.25 = 3.2$. A safety factor of 3 is recommended so that dead load stress x $1.25 \leq$ ultimate capacity $\div$ 3.

Extrapolation of Test Data. There is a great temptation to extrapolate test data to a myriad of related situations. However, caution must be used in the application

1 George Winter, "Cold-Formed Steel Structures", Structural Engineering Handbook. Edited by Edwin H. Gaylord, Jr., 1968.

2 Nilsen, p. 128.

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of this test data for several reasons:

1. The only variables tested were the web to chord connection device and the basic material (steel or aluminum).

2. The lack of usable analytical data.

3. The large number of variables.

4. The uniqueness of the test.

Some of the variables and tentative recommendations for extrapolation follow:

1. Varying spans (web depths). The Cornell tests[1] indicate a linear relationship[2] between web span and shear strength per foot. Shallower webs have a greater shear strength per foot. The variation is not extensive and this test cannot verify or disprove the relationship. The discrepancy in shear strengths between the Cornell test and this test might be explained in part by the difference in web spans.

Because California box beam depths vary considerably, some determination must be made of ultimate shear strengths. Box beam depths in current use vary from 58 in. to 128 in., the depth tested was 108 in. The tentative recommendation (which is conservative) is as follows:

a. For box beam depths 108 in. and less assume strength per foot is equal to the test results for the 108 in. box beam.

b. For box beam depths in excess of 108 in. use shear strength values for the 108 in. box beam corrected by

1 Nilsen, p. 132.

2 However, considerable "scatter" was noted.

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the factor 108/d, in which d is the depth in inches of the box beam under consideration.

2. Web thickness. The Cornell tests[1] indicate a close relationship between shear strength and the square of the web thickness. However, web thicknesses less than 16 ga. are considered unsatisfactory for box beam signs because of (1) handling problems, (2) visible surface flaws caused by the thinness, (3) an increased possibility of failure due to corrosion in the panel or the connections[2].

3. Geometrical configuration of web. The web strength and the connection strength are effected by the configuration of the web. Caution should be taken if these test results are applied to any web configuration other than the one tested.

4. Modulus of elasticity. As previously indicated a relationship between the modulus of elasticity and shear strength exists when failure is due to buckling, but the data is insufficient for a precise conclusion.

5. Connection to main chords. As will be discussed in the next section, the plug welds and the Huckbolts performed satisfactorily. These test results would be invalid for different connection types, sizes, or spacings. Further research might be of benefit in this area. For example, an increase in puddle weld or Huckbolt spacing, if feasible, might result in considerable cost savings.

1 Nilsen, p. 124

2 This statement is based in part upon the experience of one of the principle fabricators of box beam sign structures in California.

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WEB CONNECTIONS

Of concern are the web to chord angle connections and the panel to panel splice connections. As indicated on p. 22 of Appendix "A", failure occurred in various elements, but at similar loadings (i.e. there is not a clear-cut strength advantage for puddle welds or Huck-bolts). The inference, with various failures occurring in the web and the connections, is that the design is reasonably well balanced. There is no evidence to indicate that any element was under or over designed with respect to the whole.

With one exception[1] the fastening methods tested and shown on the Box Beam Sign Structure plans seem adequate with respect to type, size and spacing. No other changes are recommended without further testing.

Because the weld strength of light gage diaphragms is dependent upon the configuration of the surrounding metal, it is best established by physical testing. However, the theoretical stress carried by the weld which occurs under failure load is of interest:

<u>Steel Sign</u>. Approximate shear at failure = 96.4 K (on two panels)

$$\text{Shear stress per inch} = \frac{VQ}{I} = \frac{48.2(2.86)(52.5)}{16{,}000} = 0.45\ \text{k/in.}$$

1 As mentioned in Appendix "A", difficulty was encountered using the "puddle weld" in the fabrication of the test sign. This difficulty has also been experienced in the fabrication of signs for actual installation. The cost savings and adequacy of the puddle weld application is subject to question. It is recommended that a pre-drilled hole and plug weld be used in lieu of the puddle weld.

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Horizontal shear stress per puddle weld (or Huckbolt) =

$0.45(6) = 2.7k$

Vertical shear stress $= \frac{P}{N} = \frac{48.2}{28.5(2)} = 0.87k$ per weld

Combined vectorially, resultant shear $= \sqrt{0.87^2+2.7^2} = 2.84k$

Shear stress in weld $= \frac{V}{A} = \frac{2.84}{0.44} = \underline{6.46}$ ksi

Stress in steel Huckbolts (3/8 in. dia.) $= \frac{2.84}{0.11} = \underline{25.8 \text{ ksi}}$

Aluminum Sign. By similar calculations to those preceding, a pair of 3/8" aluminum welds carry 1.34K

Weld stress $= \frac{1.34}{2(0.11)} = \underline{6.1}$ ksi

Stress in aluminum Huckbolts (2-5/16 in. dia.) =

$\frac{1.34}{2(0.0767)} = \underline{8.7}$ ksi

DEFLECTIONS

Total cantilever deflection consists of the following elements:

1. Deflection caused by moment.

2. Slip between adjacent panels - Assumed negligible because of the continuous weld splice.

3. Relative movement between chord-angles and shear web at connections. This was assumed negligible for purposes of calculation. However, it could have been significant in the Huckbolted structures and might help explain the variance of theoretical from actual deflection for the Huckbolted steel structure depicted on Figure 7.

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4. Shear deflection - This was calculated as 42%[1] of moment deflection for the steel structure and estimated at 25%[2] of the moment deflection for the aluminum structure.

Calculated deflection considers only moment and shear. Although measured values appear to exceed calculated ones the difference is not significant from a structural view point because of the low magnitudes.

1 Arthur H. Nilsen, "Folded Plate Structures of Light Gage Steel," ASCE Transactions, Paper No. 3514, Vol. 128, 1963, p. 856.

$d_s = \frac{aM}{AG}$ where d_s = shear deflection

a = 1.3 for light gage steel

M = Moment

A = area of web

G = shear modulus taken as 11.2×10^6

2 Used for previous designs - based on Calif. Div. of Hwys., Bridge Dept. Memos to Designers, No. 16-2.

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IV. CONCLUSIONS

1. Design criteria

A. There are a great many variables determining the shear strength of a ribbed web. Extrapolation of these test results or application to dissimilar structures should be subject to extreme caution.

B. Shear capacity of the web is best described by a shear strength per foot.

C. Physical testing is still the best method for developing design criteria for the ribbed web.

D. Deflections can be reasonably approximated using conventional formulas considering moment and shear.

2. Structure performance

A. In general the structural performance of the steel and aluminum signs was satisfactory. The web to chord angle connections (plug weld or Huckbolts) and welded web splices were sufficient to develop the web's shear strength.

B. The test was not designed to test torsional strength of the box beam. However, during test loading, the structure seemed rather susceptible to a twisting action. This would suggest that the structure does not behave as a closed section in resisting torsion inducing loads.

C. Thinner web gages are not practical from the standpoint of fabrication, corrosion resistance, or appearance.

D. The web has no post buckling strength because it has little resistance to tension across the corrugations.

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V. RECOMMENDATIONS

1. For design of a 16 ga. ribbed web use the following allowable shear values[1]:

(a) Box beam depths of 108" or less (plug welds or Huckbolts)

Steel ribbed web:	1,900 lbs/ft
Aluminum ribbed web:	900 lbs/ft

(b) For box beam depths in excess of 108" use the values for 108" depth corrected by the coefficient 108/d, where d = Box beam depth in inches.

2. Use a plug weld with pre-drilled hole instead of the puddle weld for web to chord-angle connections.

3. Except for change noted in #2 the design of the box beam sign should remain unchanged.

4. Large torsional loadings on the box beam should be avoided unless internal bracing is provided.

1 These values are recommended regardless of the moment on the section under consideration.

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VI. AREAS FOR ADDITIONAL RESEARCH

1. The effect of increasing the web to chord angle connector spacing.

2. The effect, of bending moment on the shear capacity of the web.

3. The relationship of web shear strength to box beam depths.

4. The torsional strength of the box beam structure and the effect of torsional moments combined with direct shear on the webs.

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VII. REFERENCES

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FIG 1

LOAD - BENDING STRESS DIAGRAM

UPPER CHORDS 1'-6" FROM SUPPORT
STEEL STRUCTURE - PUDDLE WELDS
GAGE 1 & 2

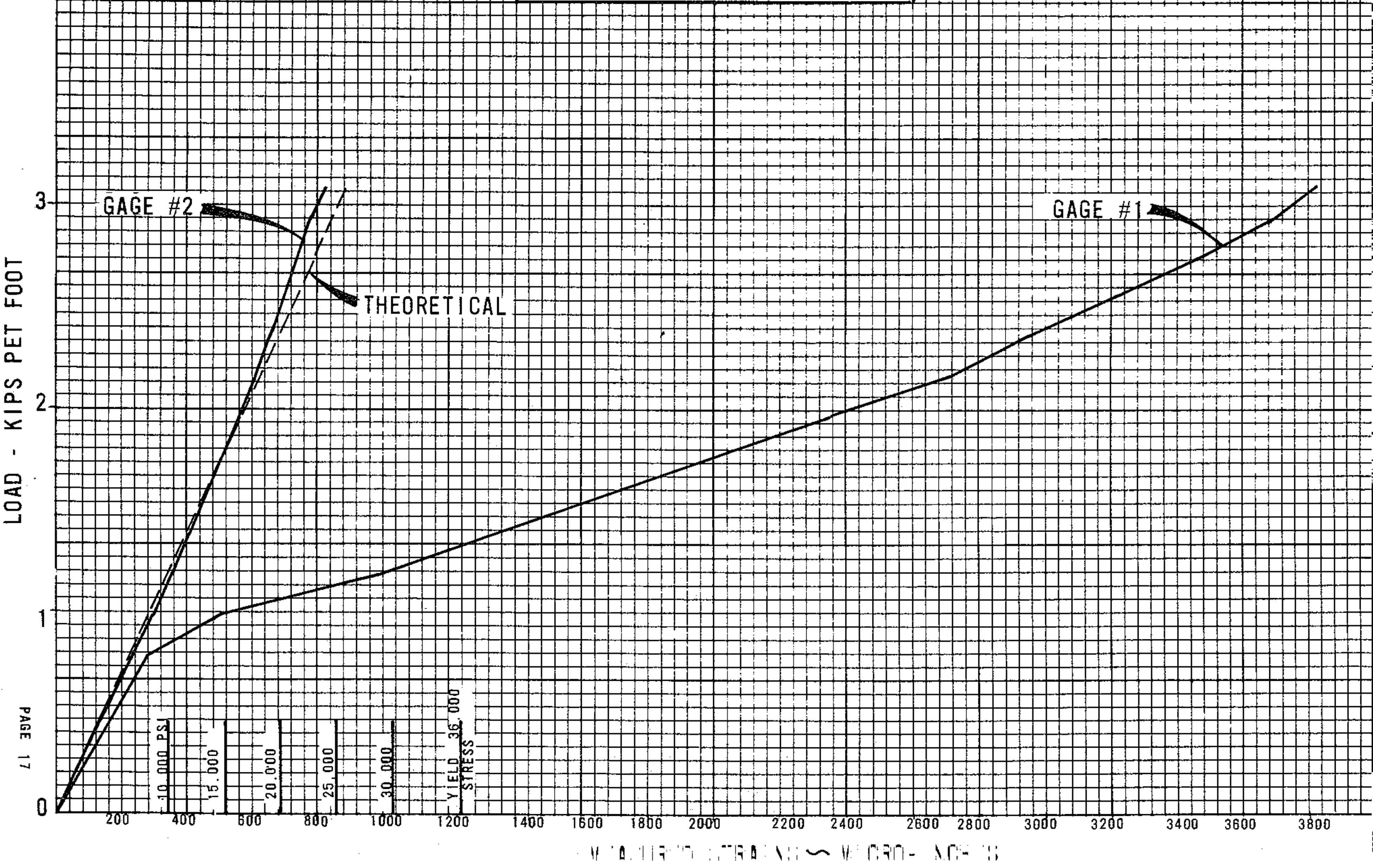

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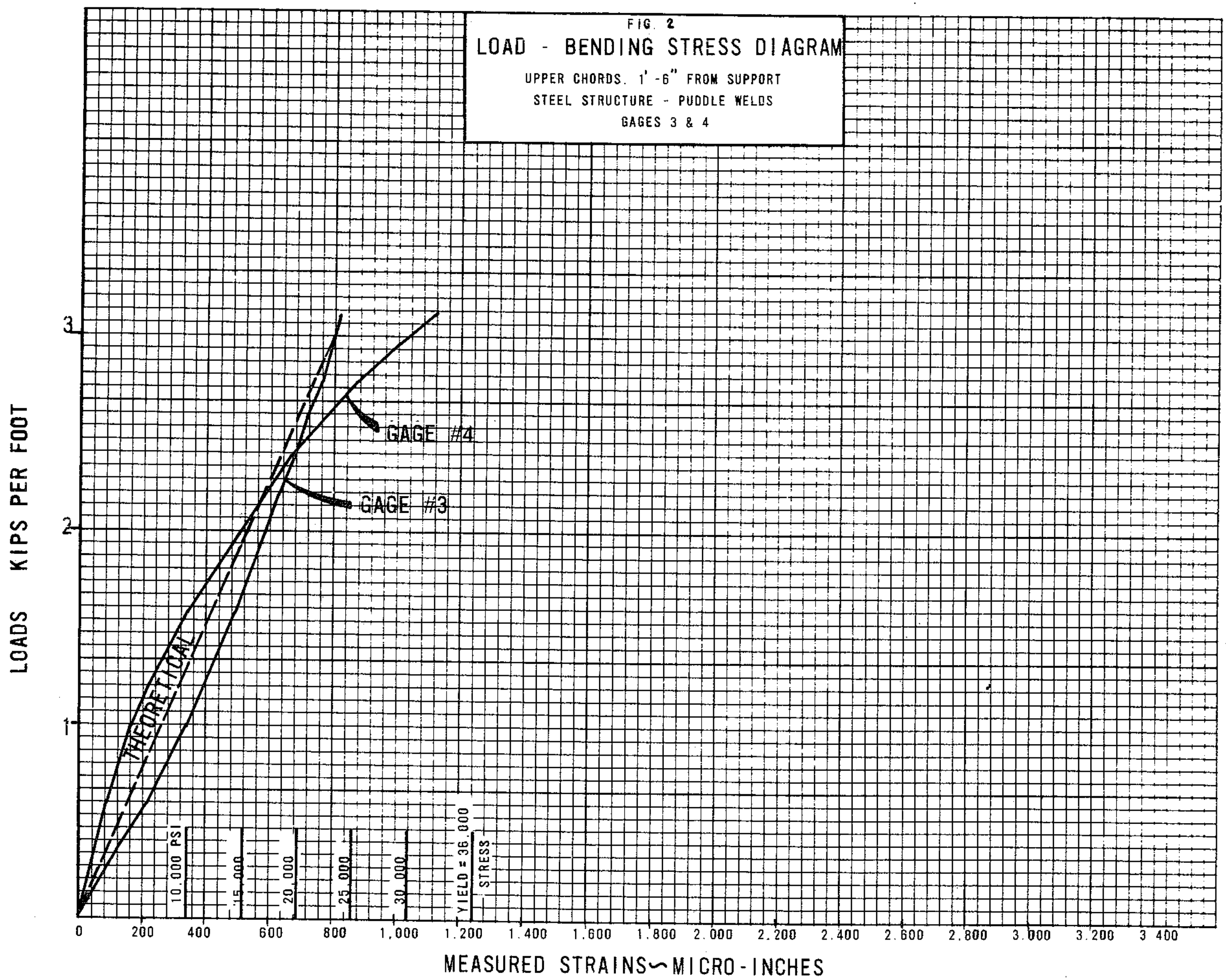
FIG. 2

LOAD - BENDING STRESS DIAGRAM

UPPER CHORDS. 1'-6" FROM SUPPORT
STEEL STRUCTURE - PUDDLE WELDS
GAGES 3 & 4



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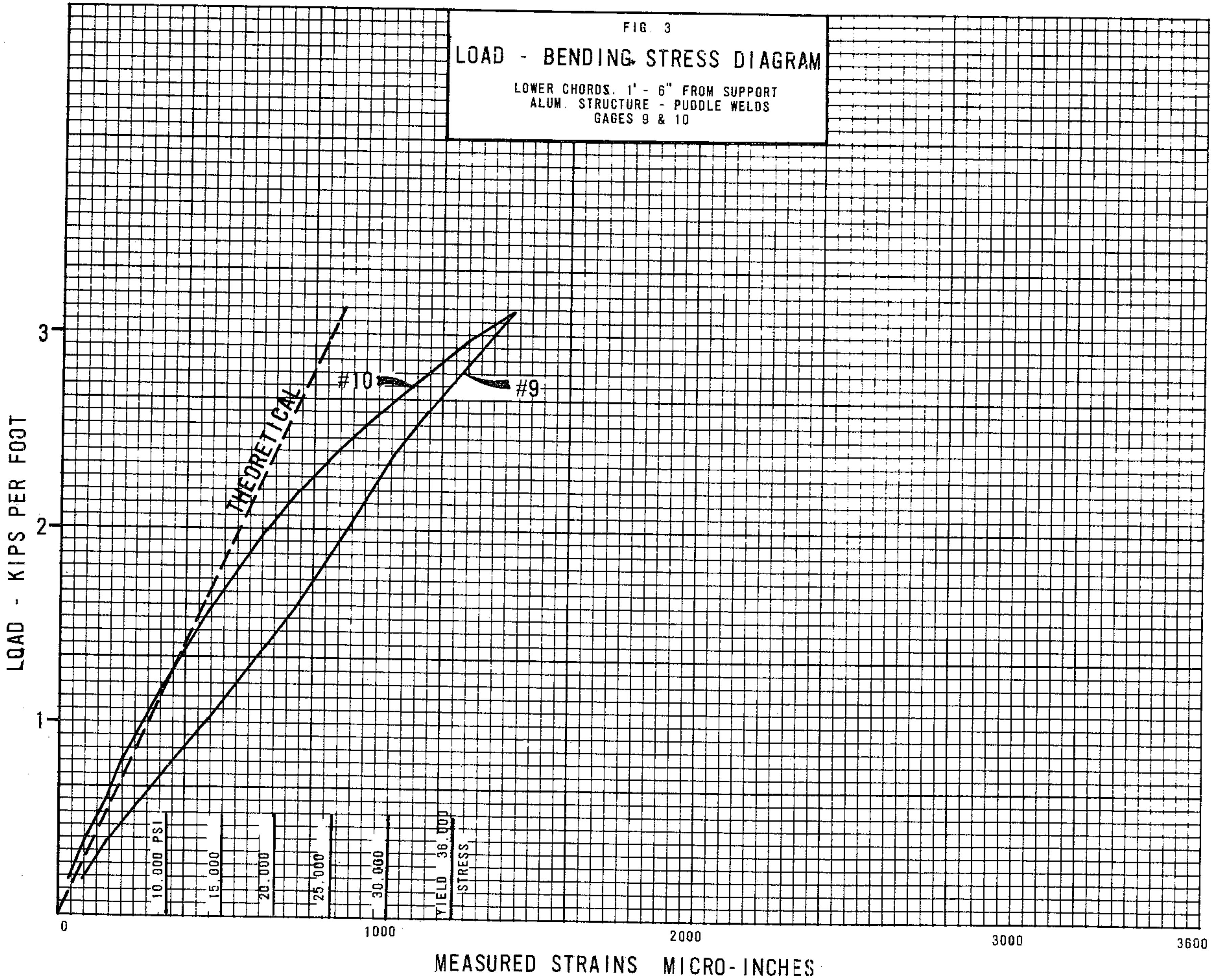

FIG. 3

LOAD - BENDING STRESS DIAGRAM

LOWER CHORDS. 1' - 6" FROM SUPPORT
ALUM. STRUCTURE - PUDDLE WELDS
GAGES 9 & 10

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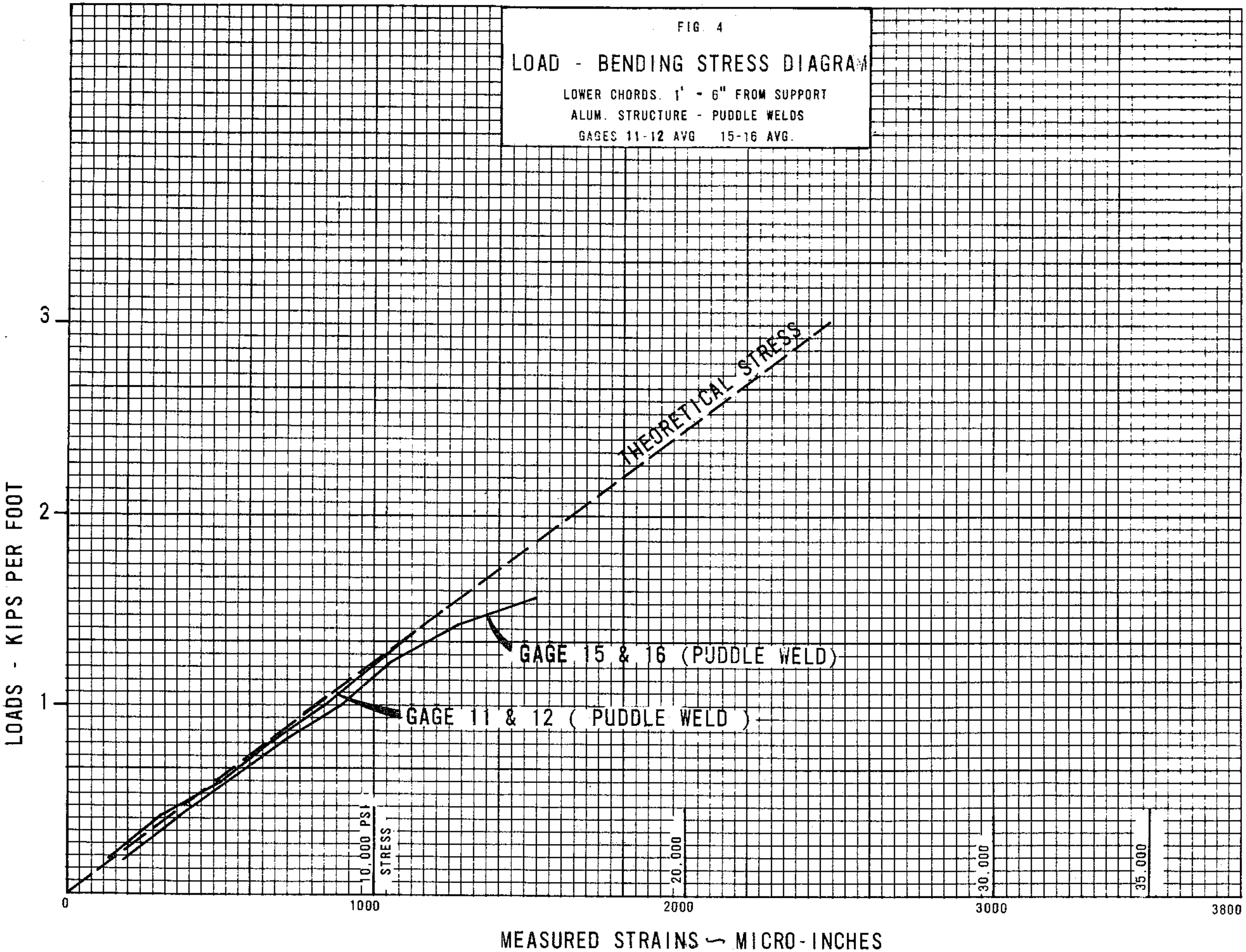

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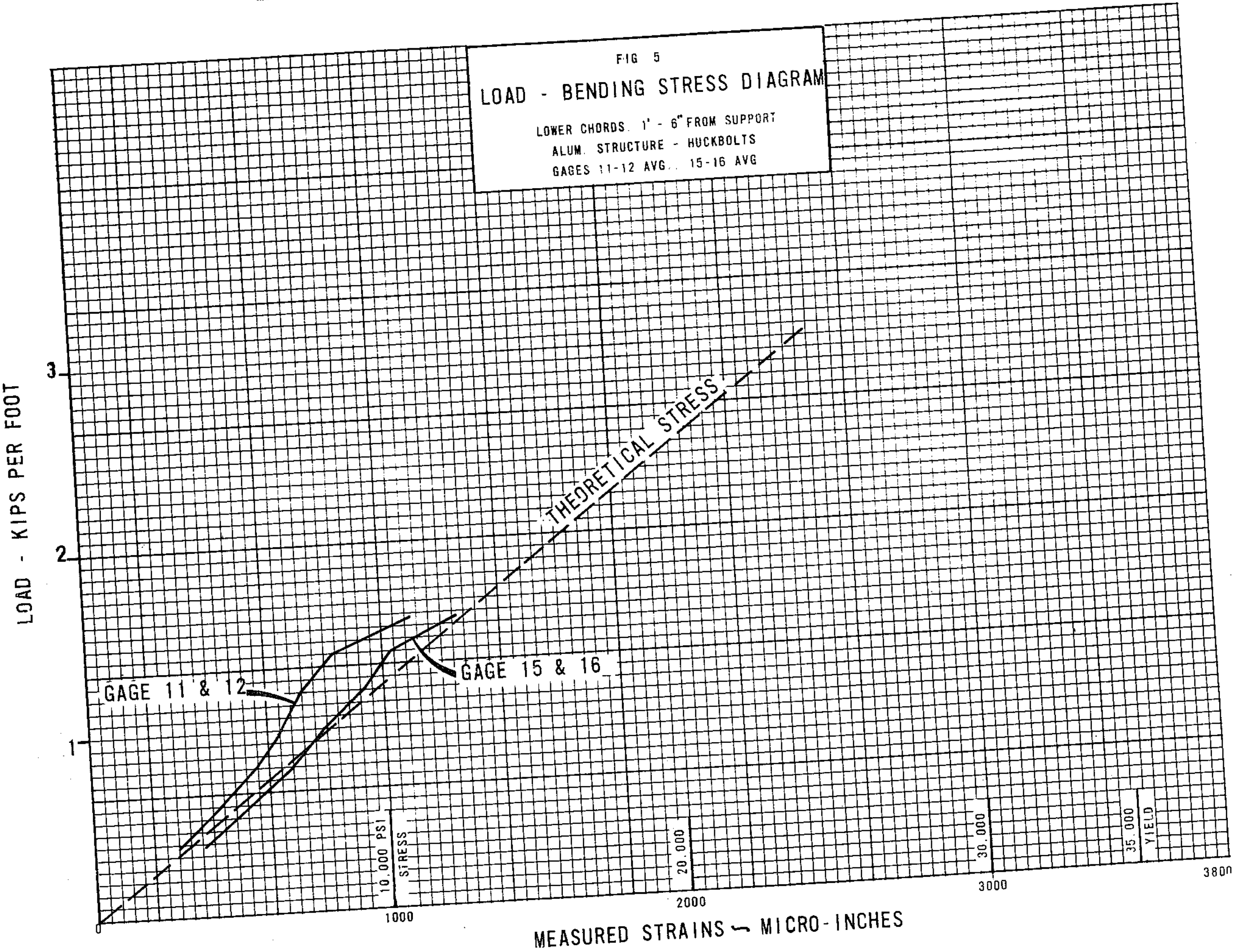

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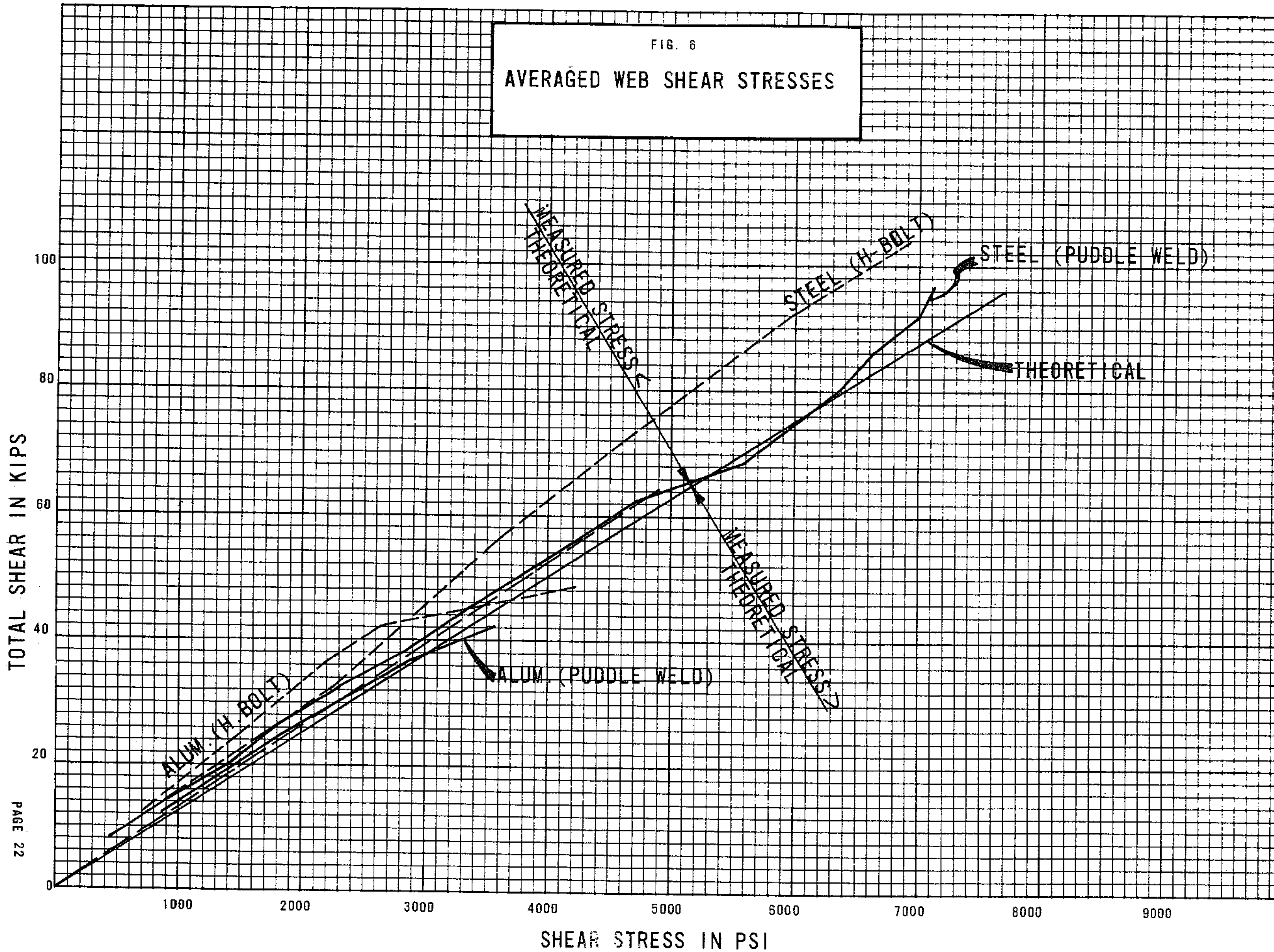

FIG. 6

AVERAGED WEB SHEAR STRESSES

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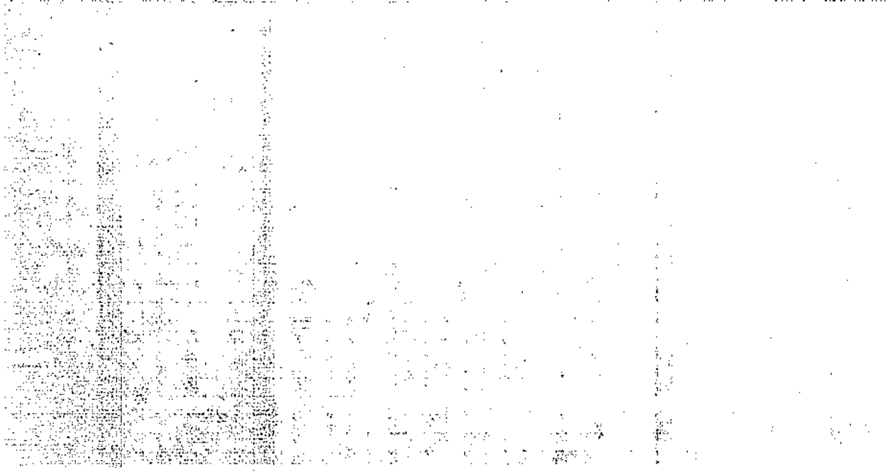

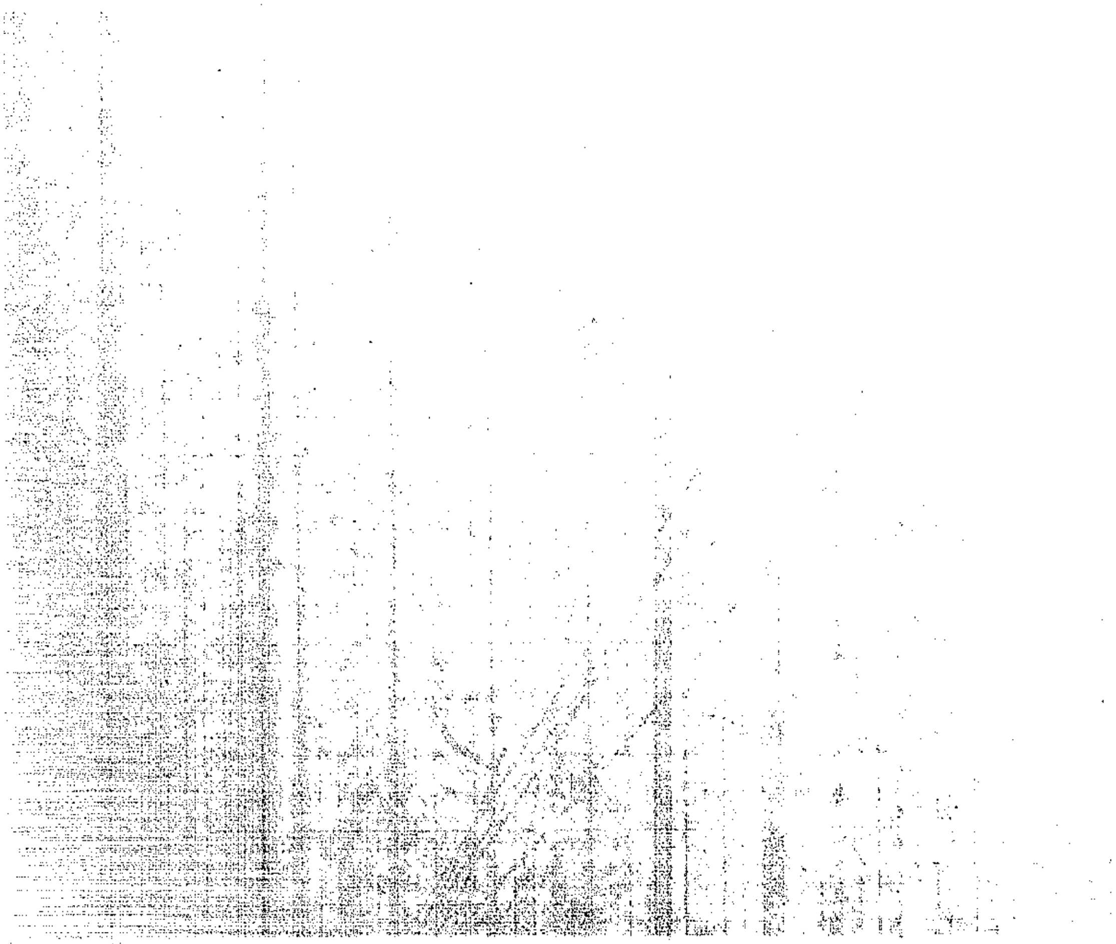

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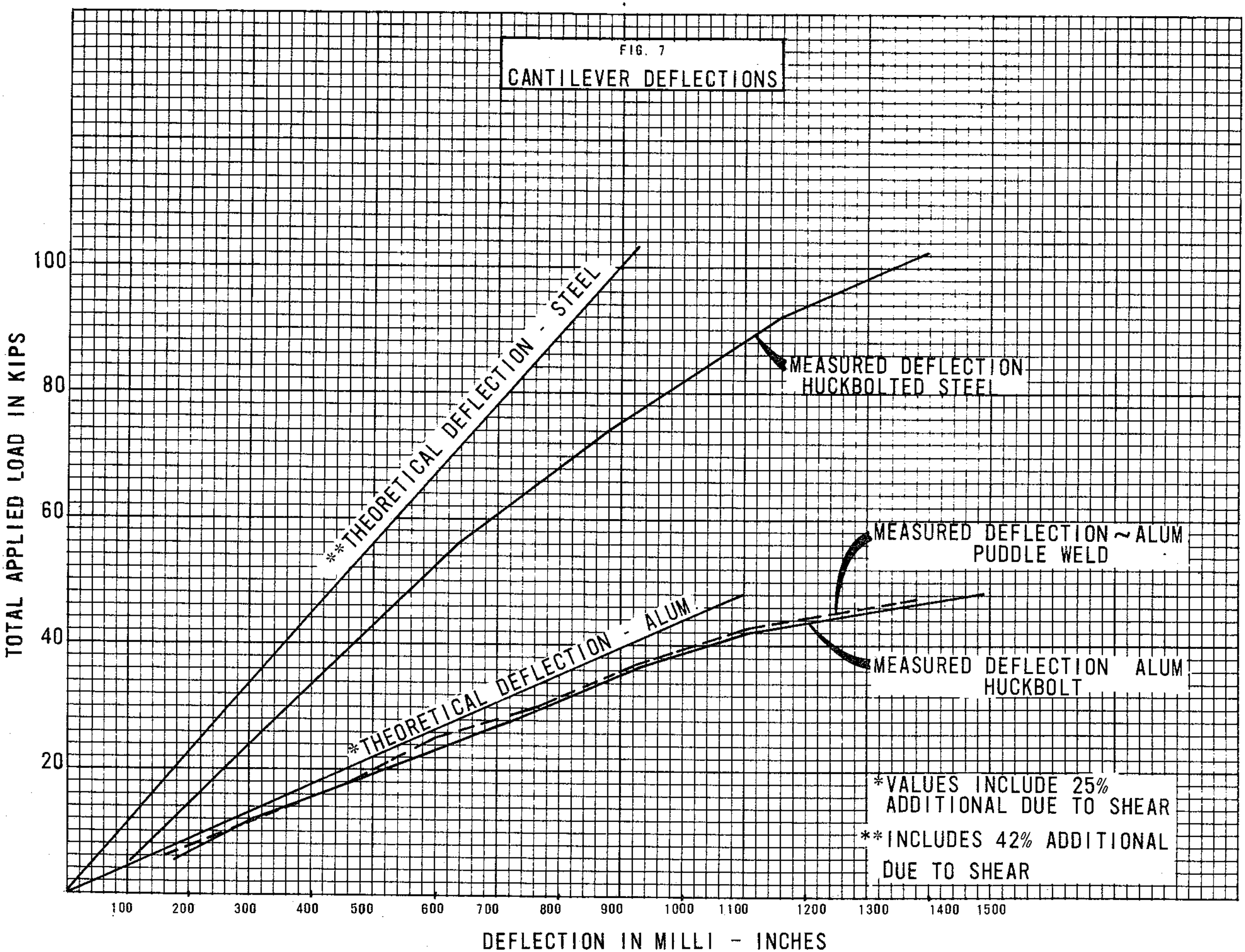

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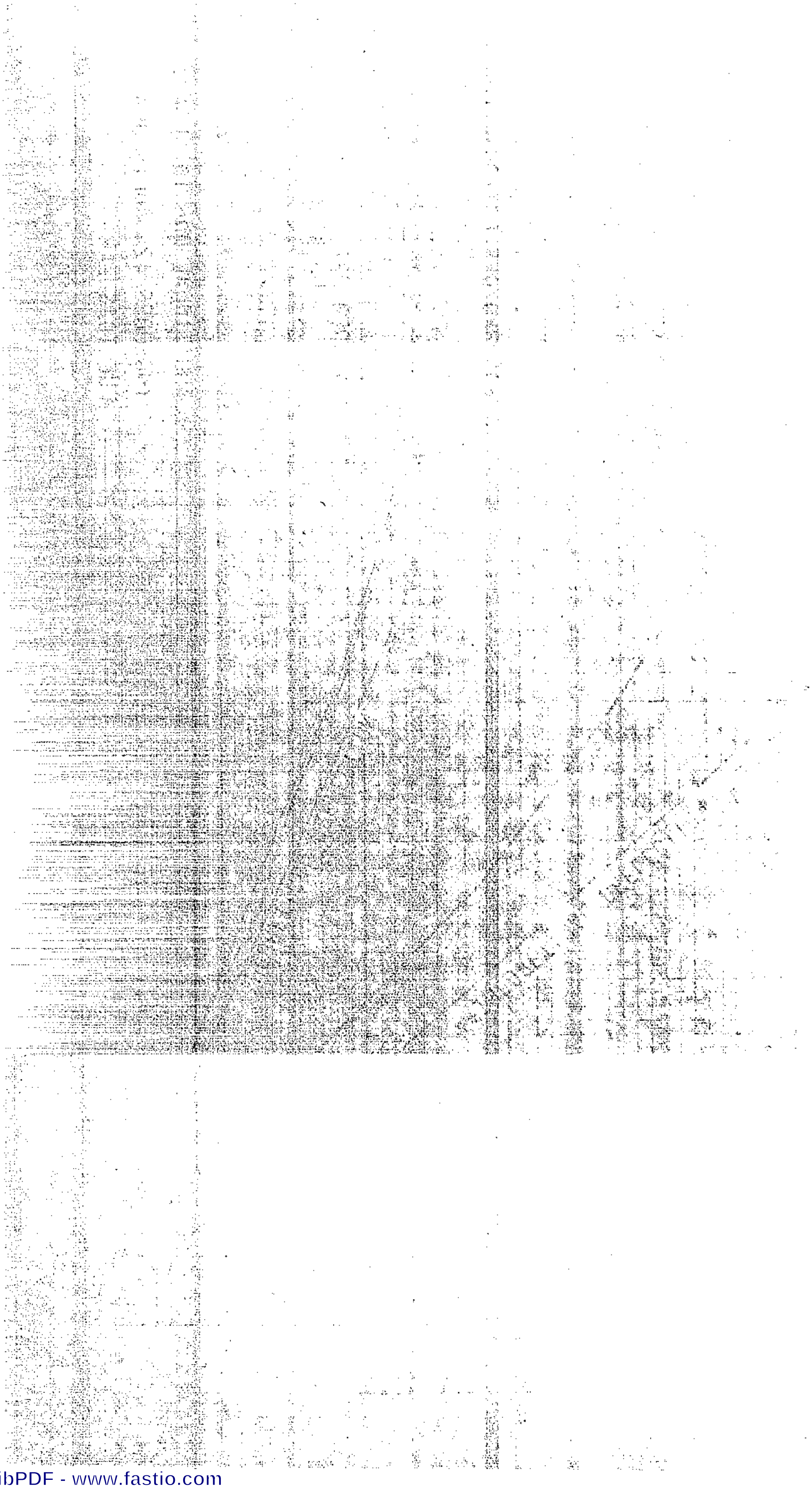

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STATE OF CALIFORNIA—TRANSPORTATION AGENCY RONALD REAGAN, Governor

DEPARTMENT OF PUBLIC WORKS
DIVISION OF HIGHWAYS
MATERIALS AND RESEARCH DEPARTMENT
5900 FOLSOM BLVD., SACRAMENTO 95819

Appendix A
to
Bridge Department's Report
"Overhead Sign Structures"
(624125)

M & R No. 36419
June 1969

Mr. J. E. McMahon
Assistant State Highway Engineer, Bridges
California Division of Highways
Sacramento, California

Attention: Mr. G. D. Mancarti

Dear Sir:

Submitted for your consideration is a report of

FULL SCALE DESTRUCTIVE TESTING

OF

TWO BOX BEAM OVERHEAD SIGN STRUCTURES

ERIC F. NORDLIN
Principal Investigator

W. H. AMES, R. N. DOTY, and M. H. JOHNSON
Co-Investigators

Assisted By
E. R. Post
V. C. Martin
K. Cook
S. Dukelow

Very truly yours,

JOHN L. BEATON
Materials and Research Engineer

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ABSTRACT

REFERENCE: Nordlin, E. F. and Ames, W. H., "Full Scale Destructive Testing of Two Box Beam Overhead Sign Structures", State of California, Highway Transportation Agency, Department of Public Works, Division of Highways, Materials and Research Department. Research Report 36419, June 1969.

ABSTRACT: This is a report of full scale testing to ultimate capacity under a uniformly distributed load of two 60-ft box beam overhead sign structures of a design recently adopted, primarily because of aesthetic considerations, by the California Division of Highways. This design has been utilized on a trial basis at selected locations on California freeways.

One test structure was fabricated completely of steel and the other of aluminum. One 30-ft cantilevered end of each structure utilized plug welds for the web-to-flange connection while the other end used Huckbolts. The steel structure failed at about 175% of the designers' predicted failure load of 1.9 kips per lineal foot. The aluminum structure failed at 83% of that value. This report will be included as Appendix A of the California Division of Highways' Bridge Department's final report entitled "Overhead Sign Bridge". This Bridge Department report will contain an analysis of the data and an evaluation of the sign structure design.

KEY WORDS: Aesthetics, aluminum, bolted joints, box beams, fasteners, load tests, riveted joints, sign structures, testing, welded joints.

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The opinions, findings and conclusions expressed in this report are those of the authors and are not necessarily those held by the Bureau of Public Roads.

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TABLE OF CONTENTS

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I. INTRODUCTION

The current emphasis on aesthetics in highway design has directed attention to the desirability of an alternative to the open truss sign bridge frames that have been standard for many years on California freeways. Accordingly, a special committee consisting of representatives of the Traffic, Bridge, and Maintenance Departments of the California Division of Highways recommended adoption of the more aesthetic concept of an enclosed box beam section.

The proposed design, utilizing side webs of vertically ribbed sheet metal connected to the flanges by puddle welds or Huckbolt fasteners, was subsequently incorporated into contract design plans by the Division of Highways' Bridge Department. However, these design features had not previously been used in large box beam structures, and no information regarding the load carrying capacity of such a design was available. In an effort to further develop this design and assure both economy and safety, the Bridge Department initiated a research project that consisted of fabricating and testing to failure two full size sign structures, one of steel and one of aluminum.

The following characteristics were considered to be of particular importance in planning and executing the test program:

1. The load carrying capacity of the structures, as compared to design computations.

2. The shear capacity of the web section.

3. The effectiveness of the two fastening methods.

4. The relative load carrying capacity of steel and aluminum in this application.

This report covers the work performed by the Materials and Research Department in arranging for the fabrication of the sign structures, in designing and erecting the supporting and loading apparatus, in instrumenting the sign structures, and in acquiring and processing the test data. Data compiled in the form of computer printouts and digital tapes were furnished to the Bridge Department prior to publishing this report. Analysis of the test data and evaluation of the design concept will be performed and reported by the Bridge Department in their research report number 624125 titled "Overhead Sign Bridge".

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II. THE SIGN STRUCTURES

The type of sign structure selected for this study was a single post "butterfly" (balanced cantilever) design. This type was chosen as the most critical case because the maximum shear and bending moment both occur at the same location (i.e., at the sign support). Both a steel structure and an aluminum structure were tested; their dimensions and configuration were essentially the same. Over-all dimensions were 60 ft x 9 ft x 2 ft. The box beam section was composed of ribbed sheet metal side webs and trussed flanges reinforced by 4 transverse diaphragms (Figures 1 and 2).

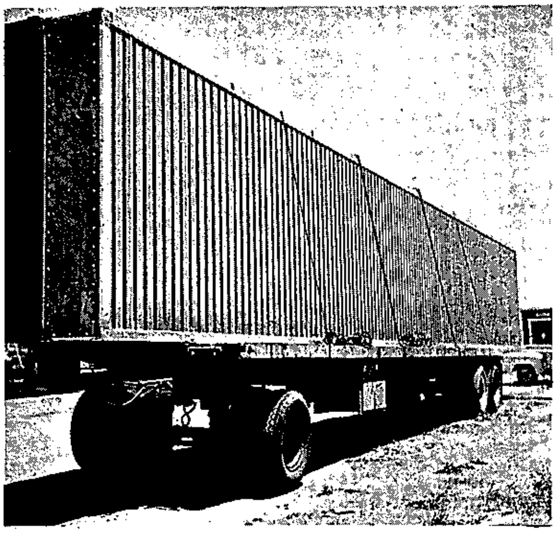

Figure 1

ALUMINUM SIGN STRUCTURE

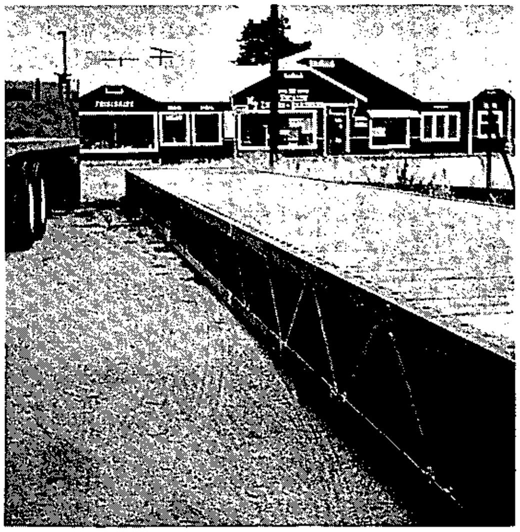

Figure 2

STEEL SIGN STRUCTURE

For one half (30 ft) of each sign frame, plug welds were used to attach the ribbed sheet metal to the top and bottom chord angles and to the interior post diaphragm angles; for the other half, Huckbolts were used. The steel structure had one web-to-chord fastener at the top and bottom of each web face panel whereas the aluminum structure had two at each location. The indented trapezoidal sheet metal ribs were spaced on 6-in. centers and were 1½ in. deep, 2-1/8 in. wide at the face, and 3/4 in. wide at the indented base; face panels were 3-7/8 in. wide.

"Huckbolt" is a patented fastening system which uses a round headed pin and a swaged locking collar. Beyond the grip of the pin there are two series of annular grooves, one for locking and one for pulling, which are separated by a deeper breakneck groove. The pin is inserted into the work from one side and the locking collar is slipped onto the pin from the other side. A fastening tool, when

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fitted over the pin and activated, grips the pulling grooves at the end of the pin, pushes a swaging anvil against the locking collar, and causes the collar to bear against the work. As the tensile force in the pin is increased, the pieces are clamped firmly and the swaging anvil swages the collar into the locking grooves. The pin then fractures at the breakneck groove, and the pulling end is discarded.

Although the contract specified spot puddle welds as the second method for fastening the web to the flanges, the fabricator experienced so much difficulty in producing welds of the required strength that he was permitted to use plug welds as a substitute. Punched hole diameters for these welds were 7/16-in. for the aluminum sheet metal and 9/16-in. for the steel.

The top and bottom flanges of the box beam structures were Warren trusses composed of two 4 x 4 x 3/8 angle chords braced by $1\frac{1}{2}$ x $1\frac{1}{2}$ x $\frac{1}{4}$ angles at 45° (Figure 2). Reverse patterns (top vs. bottom) were utilized. Additional transverse stiffening was provided by two end plate diaphragms and two interior or post diaphragms. The post diaphragms were located 15 in. to the left and right of the center of the structure. The end diaphragms were single $\frac{1}{4}$ in. plates ($21\frac{1}{4}$ in. x 106 in.) attached to 4 x 4 x 3/8 angles which in turn were welded to the chord angles at each corner and to the web along each side. The post diaphragms consisted of three $\frac{1}{4}$ in. plates attached to 3 x 3 x $\frac{1}{4}$ angles (Figures 3 and 4). The top plate was 6 in. x $19\frac{1}{4}$ in., the middle plate was 10 in. x $19\frac{1}{4}$ in., and the bottom plate was 13 in. x $19\frac{1}{4}$ in. The ends of the top and bottom angles were welded to the chord angles. The side angles were attached to the indentation base of a side web rib. Figures 3 and 4 show a post (interior) diaphragm of the aluminum structure with the original aluminum post bolted in place.

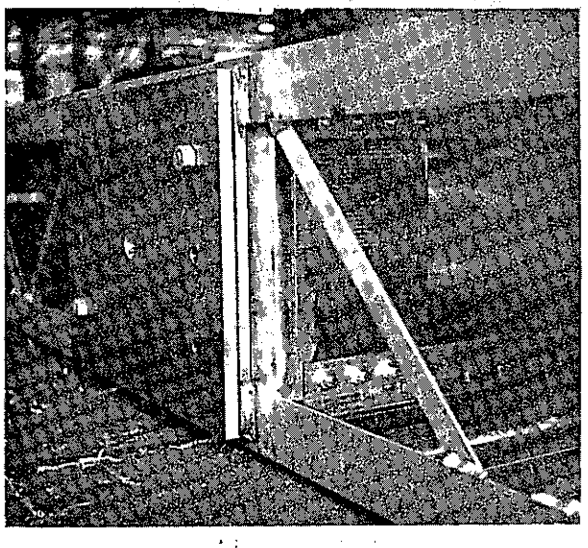

Figure 3

ALUMINUM STRUCTURE POST BASE AND LOWER DIAPHRAGM PLATE

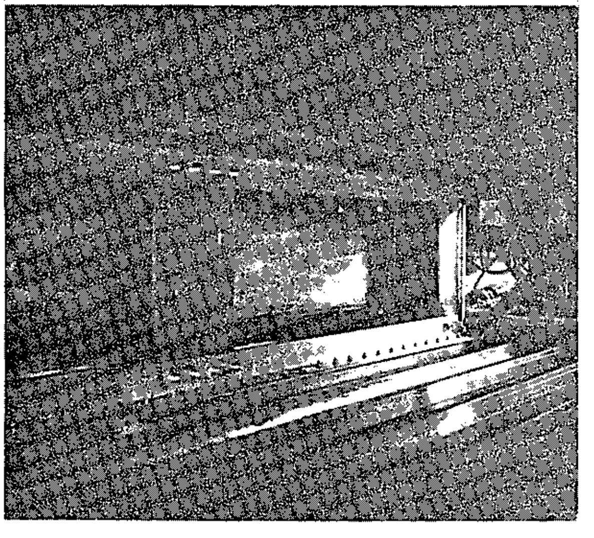

Figure 4

ALUMINUM STRUCTURE POST AND UPPER DIAPHRAGM PLATES

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Fabrication of the structures was contracted, after competitive bidding, to California Blowpipe & Steel Co. of Escalon, California, according to plans prepared by the Bridge Department and specifications prepared by the Materials and Research Department. Inspection was performed by personnel of the Materials and Research Department's Sacramento Inspection Office and the District 10 Materials Department.

The steel sign structure contract specified that all plates and shapes except the ribbed sheet metal conform to ASTM Designation: A-36; that the ribbed sheet metal be fabricated from 16 gage (0.60 in.) uncoated carbon steel sheet conforming to ASTM Designation: A-245, Grade C; and that nuts and bolts be high strength conforming to ASTM Designation: A-325.

The aluminum structure contract specified 0.063-in. thick sheet metal for the webs. All the materials used conformed to the respective requirements of the ASTM designations for the aluminum alloy and heat-treatment listed in the following table:

Item	ASTM Designation	Alloy & Heat No. Treatment
Structural Shapes	B-308	6061 - T6 or 6062 - T6
Ribbed Sheet Metal and Plates	B-209	6061 - T6
Bolts	B-211	2024 - T4
Washers	B-209	2024 - T4
Nuts	B-211	6262 - T9

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III. THE TESTING SUPPORT STRUCTURE AND LOADING SYSTEM

The basic requirement for the test apparatus was that it effectively simulate a uniformly distributed dead load of sufficient magnitude to fail the structure. The test apparatus was designed for a load of 4 kips per lineal foot, which was just over twice the bridge designers' predicted failure load for the steel sign (1.9 kips per lineal foot). Another consideration was avoiding damage to one end of the structure while testing the other.

The method selected was to support the sign structure in upright position at the center only, to use hydraulic jacks to pull down on one end, and to use tension braces to resist the moment reaction in the support post (Figures 5 and 6). This approach, although somewhat complex, eliminated some of the variables that would be involved if the structures were tested in any other orientation. A drawing of the support structure is included as Exhibit 1 of the Appendix.

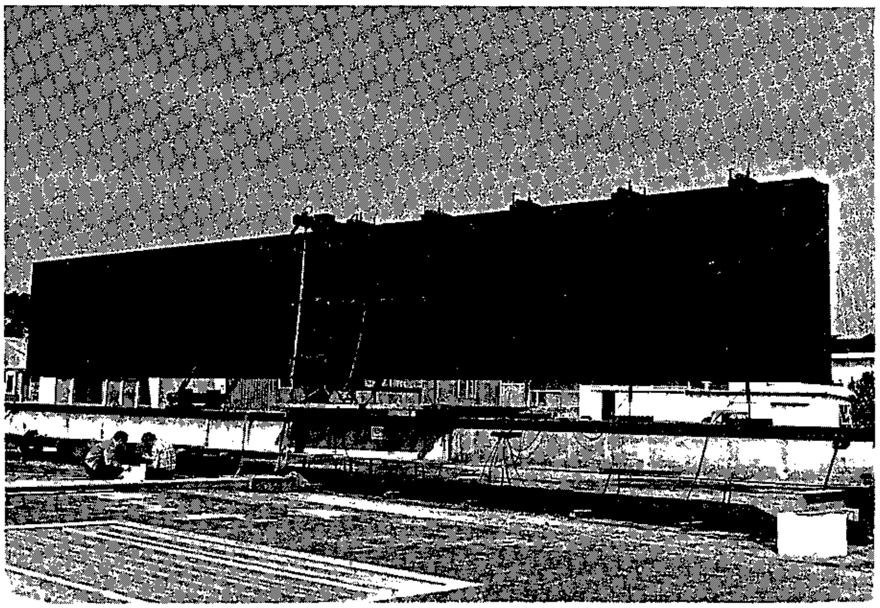

Figure 5

WELD CONNECTED END OF STEEL STRUCTURE PREPARED FOR INITIAL TESTING

(SIDE VIEW)

Two concrete pedestals supported each lateral 10WF33 beam. They were 2-ft square and spaced 9-ft apart (center to center) to provide resistance to lateral wind loads. The two 10WF33 beams were spaced 63-ft apart (center to center) as end supports for two 30WF172 beams, which were spaced 27-in. apart (center to center) to provide 1 ft of inside clearance for test apparatus. The 30WF172 beams were also tied to the 10WF33 beams with 45°

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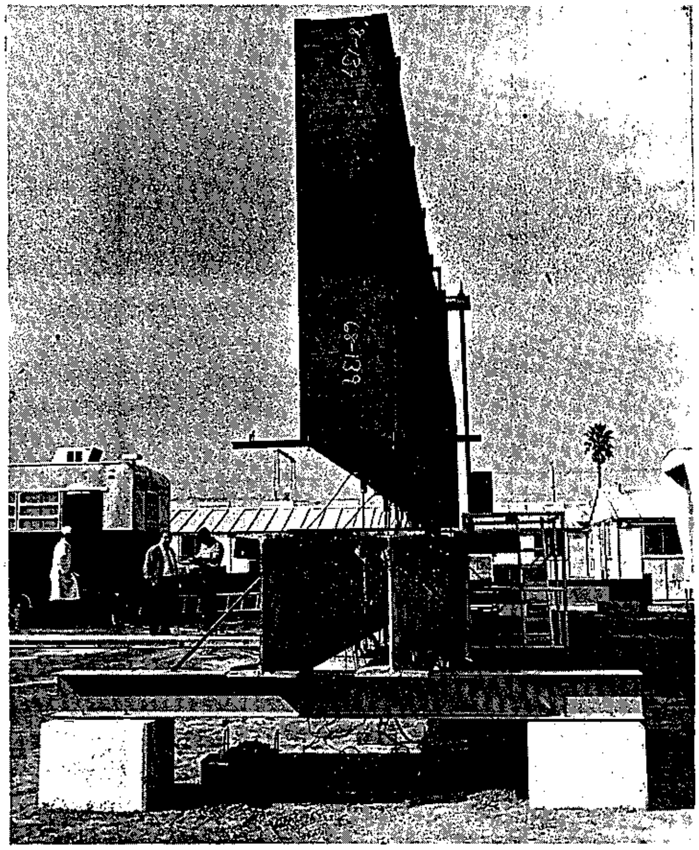
wind braces of 1 in. diameter AISI 1018 cold drawn steel bars. Elastomeric bearing pads were used under all four beams. The 1-in. diameter brace bars were welded to the top flanges of the 10WF33 beams while all other connections were bolted. The maximum sign frame end deflection was estimated to be only about 5 in. whereas the clearance between the sign frame and the support structure was slightly more than 20 in.



WELD CONNECTED END OF STEEL STRUCTURE PREPARED FOR INITIAL TESTING

(END VIEW)

Two 2½-in. diameter rods of AISI 1018 cold drawn steel (54 ksi yield strength) were utilized as moment resisting tension braces (Figures 7 and 8). The upper brace pin was an 8-in. diameter round of AISI 4041 steel hot rolled and heat treated to provide a minimum uniform yield strength of 85 ksi. The lower brace pin was an 8-in. diameter round of AISI 1042 hot rolled steel having a minimum uniform yield strength of 59 ksi. Both brace pins were 58 in. long.

The brace rods were fabricated in two pieces with a sleeve nut coupling to facilitate handling during erection and dismantling. The moment resisting brace system was designed for ready dismantling between tests so that the sign structure could be removed for turning and replacing. Provisions were also made to load from either end of the support structure should that become necessary.

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A longer and more rigid support post than that used in actual field installations was required to satisfy test loading requirements. To achieve the necessary strength with a cross section which could not be greater than 12 in. x 12 in., 1½-in. plates of ASTM A-441 steel (46 ksi yield strength) were used for the column walls. ASTM A-36 steel was used for the remaining post components. The post was mounted on the 30WF172 beams midway between the beam supports. Bearing stiffeners were welded to the webs of the beams directly beneath the post.

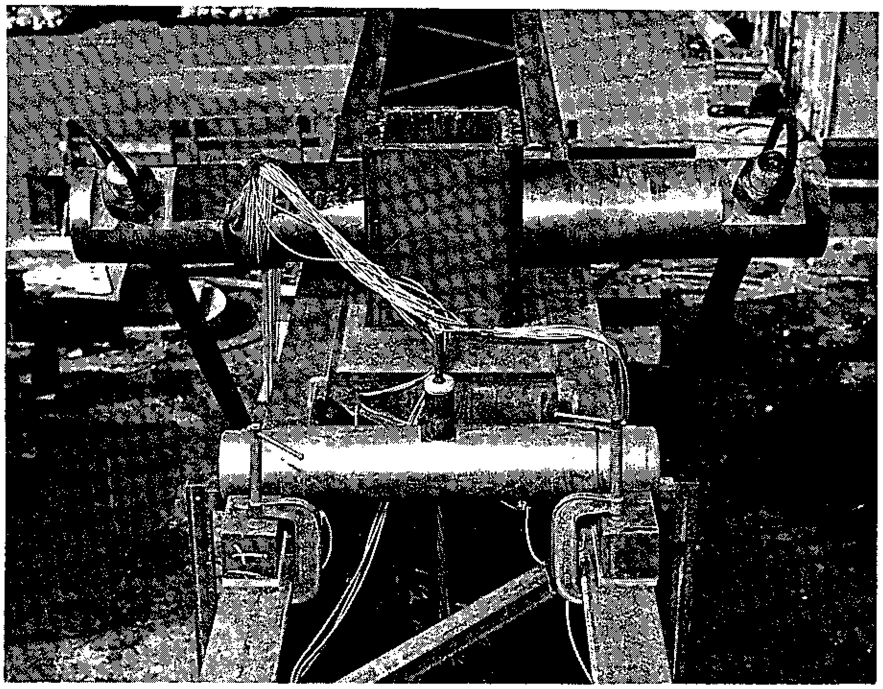

Figure 7

COLUMN TOP, UPPER BRACE CONNECTION AND ORIGINAL LOAD DISTRIBUTION MEMBERS

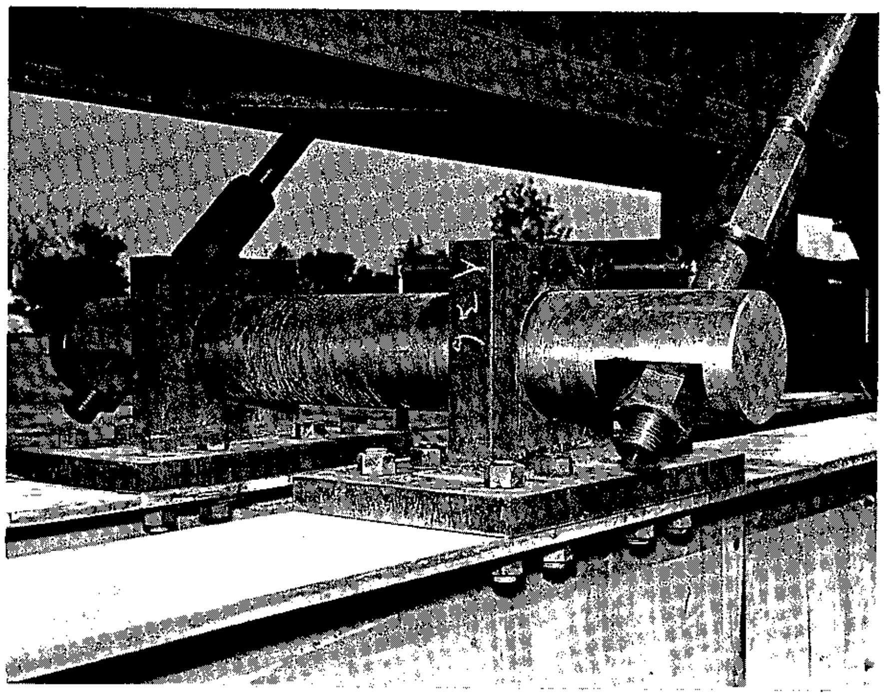

Figure 8

LOWER BRACE CONNECTION

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Originally, 3 in. x 3 in. x 12 in. bearing blocks were used in pairs to transmit testing loads from a 4½-in. diameter pin to the top flanges, as shown in Figure 7. However, the initial testing in October and November 1968 showed that significant local stress concentrations were being applied to the chord angles of the top flange and were inducing substantial outward buckling of the side webs even at low load levels. Consequently, the load distribution system was redesigned (see Figures 9 and 10 and Exhibit 2 in the Appendix). The number of bearing members was doubled, and the dimensions of the bearing surface per member was lengthened from 12-in. to 30-in. and narrowed from 3-in. to 1-in. The original bearing blocks were trimmed to a width of 2-in. and welded to the middle of 38-in. lengths of 5 I 10 bearing distribution beams. These in turn transmitted the load to the midpoint of 30-in. lengths of 14 B 17.2 bearing members by means of 2-in. x 3-in. pads of 1-in. thick layered fabric bearing material. The load was transmitted to the sign structure along the full 30-in. length of the 14 B 17.2 beam sections by 1-in. wide strips of 1-in. thick elastomeric bearing material. Figures 9 and 10 and Exhibit 2 in the Appendix show the arrangement of the members. Note that the 1-in. wide elastomeric bearing pads were located next to the outside edge of the chord angles. This placed the centerline of the load very close to the web-to-angle leg connection plane.

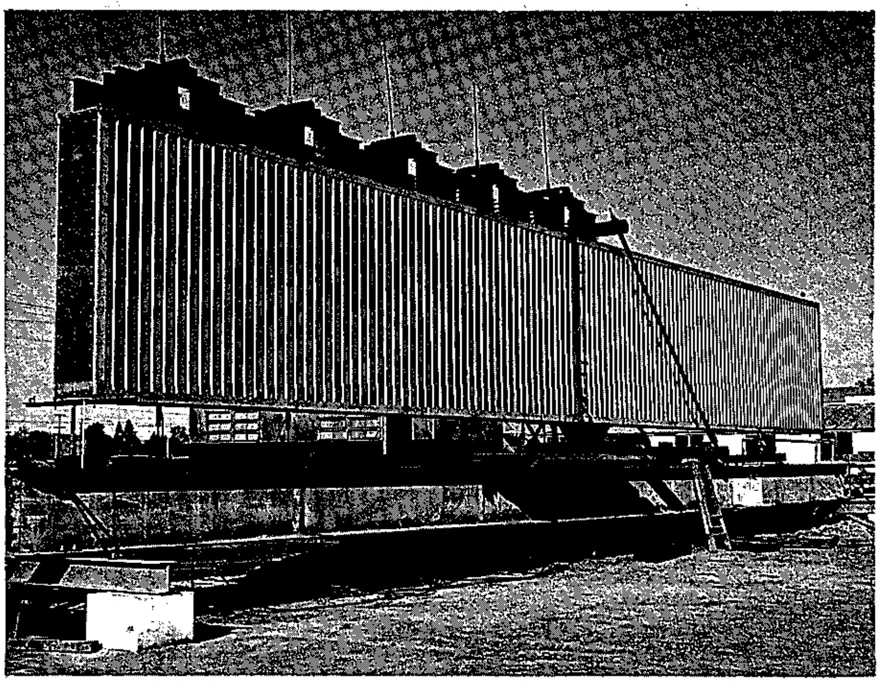

Figure 9

REVISED LOAD DISTRIBUTION APPARATUS FROM NORTH SIDE OF ALUMINUM SIGN STRUCTURE

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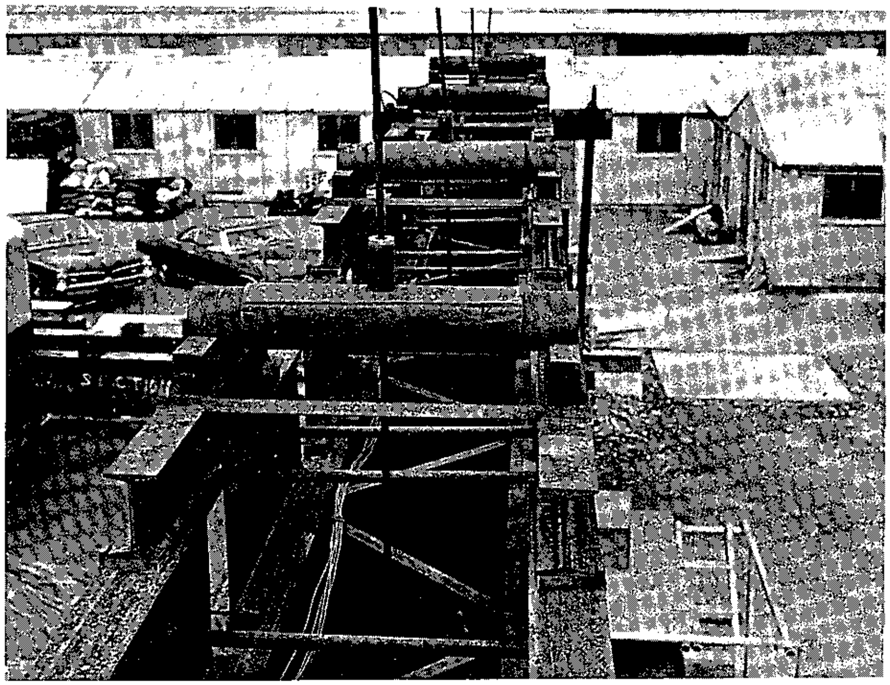

Figure 10

REVISED LOAD DISTRIBUTION APPARATUS FROM EAST END OF STEEL SIGN STRUCTURE

Five 60-ton capacity center-hole hydraulic jacks with an extension range of 10 in. were located at the center of each 6-ft increment from the center to the end of the sign structure; i.e., at 3 ft, 9 ft, etc., from centerline. The jacks were suspended from 42-in. sections of 8[22.8 channel, which were in turn bolted to the top flanges of the 30WF172 beams (Figure 11). The jacks were calibrated in a universal testing machine to determine their relative efficiencies. This calibration indicated that all five jacks could be operated from one pressure source without significant load differences. A heavy duty, electric powered hydraulic pump was connected to a pressure manifold equipped with a pressure gauge and shutoff valve for each jack line (Figure 12).

One-half inch diameter, 7 wire, high-strength steel strand was used to connect the jacks (Figure 11) to the 4½-in. diameter pins on top of the sign structure loading apparatus (Figure 10). Friction gripping chucks, which grip the strand in one direction only, were used (1) to transmit jacking loads into the strands, (2) to attach the two sections of strand to the tension load cells (positioned just below the bottom flange of the sign structures), and (3) to transmit the force in the strand onto the 4½-in. diameter pins atop the load distribution assembly.

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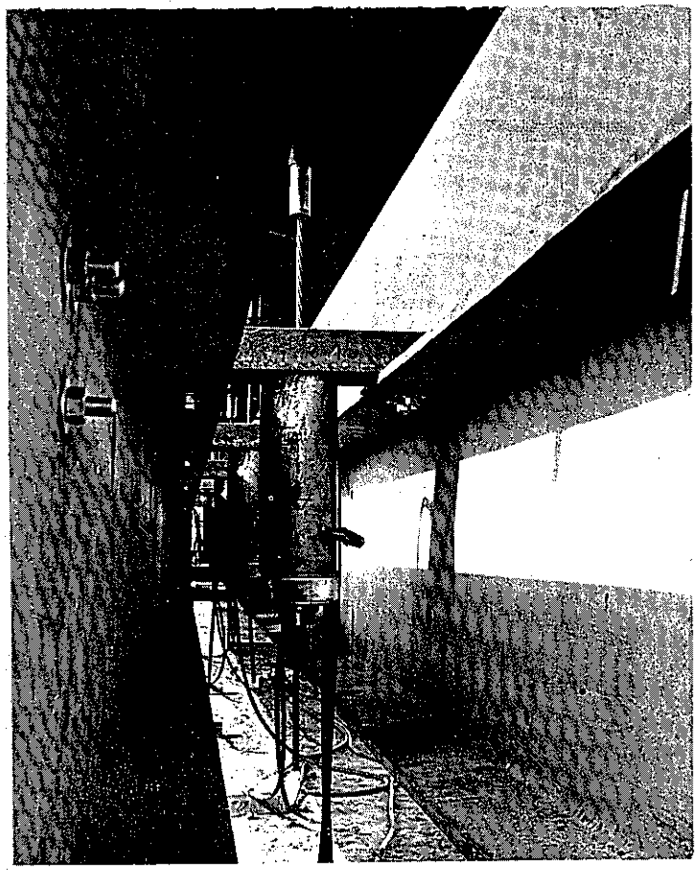

Figure 11

SIXTY TON
HYDRAULIC
JACKS

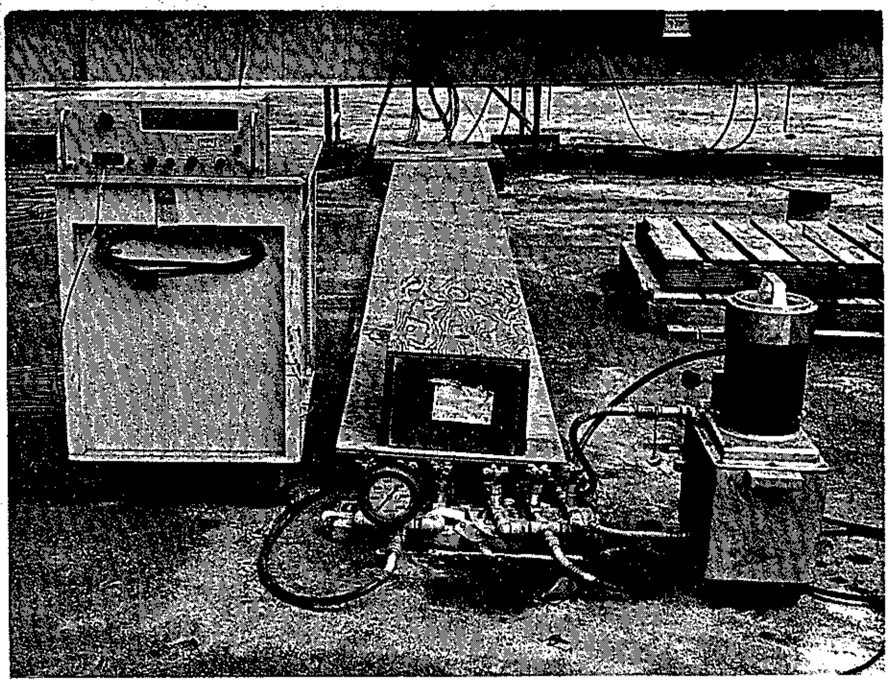

Figure 12

DIGITAL VOLTMETER,
PRESSURE MANIFOLD
WITH GAUGE AND
SHUTOFF VALVES,
AND HYDRAULIC PUMP

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IV. INSTRUMENTATION AND DATA ACQUISITION

The objectives of the sign structure instrumentation were to determine (a) the stress patterns and lateral deflection of the side webs under load, (b) the stress levels and distribution in the flange chord angles, (c) the vertical deflection of the sign structure, (d) the magnitude of applied loads, and (e) the effectiveness of the load distribution apparatus. Instrumentation location and usage per test are shown in Exhibits 3 and 4 of the Appendix.

All data was initially retrieved and recorded as voltages by data acquisition systems housed in an instrumentation trailer at the test site (Figure 13). The recorded voltages were then converted by computer processing into strains, stresses, loads, deflections, calibration values, and post testing zero changes. BASIC language programs were used for all test data. The program, as revised for the last two tests, is shown in the Appendix as Exhibit 9 and one of its load run printouts is shown as Exhibit 10. A data flow chart and a data processing equipment list are included in the Appendix as Exhibits 5 and 6.

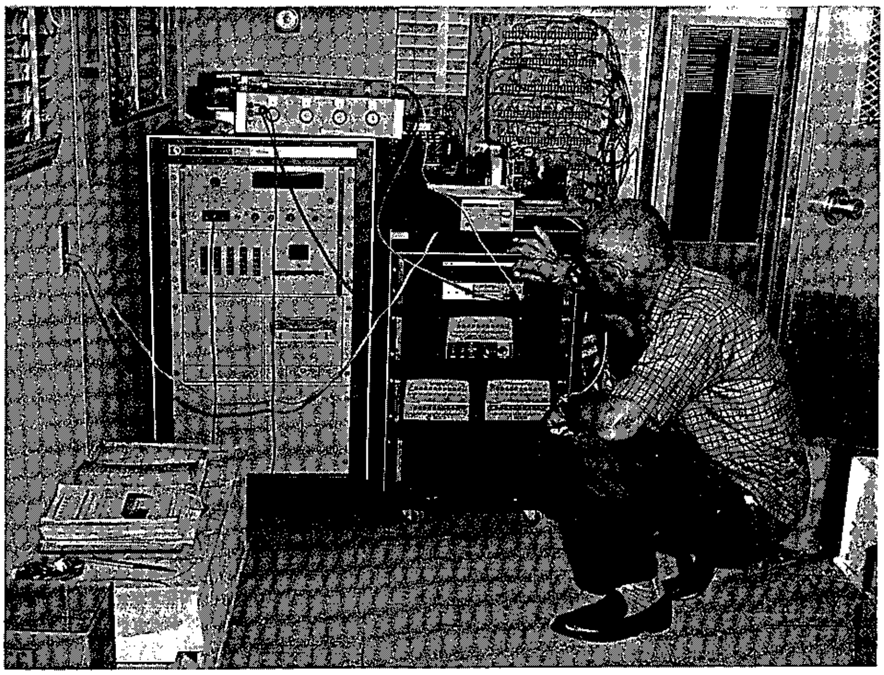

Figure 13

DATA ACQUISITION SYSTEMS

The initial instrumentation for each side web consisted of 3 (BLH SR4) 45° rosette strain gages in a vertical row

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located 18 in. from centerline on the web face panel immediately outside the post diaphragm (Figure 14) and 3 pair of (Bourns) linear potentiometers ("pots") in a horizontal row at mid depth of the side web.

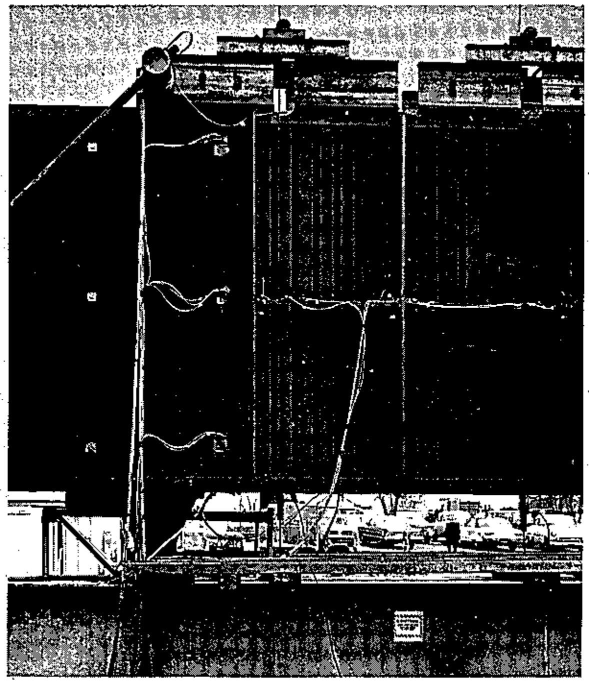

Figure 14

WEB INSTRUMENTATION FOR PLUG WELD SIDE OF STEEL STRUCTURE

One "pot" in each pair measured lateral web deflections (buckling), and the other measured rib distortion horizontally across the web face (Figures 15 and 16).

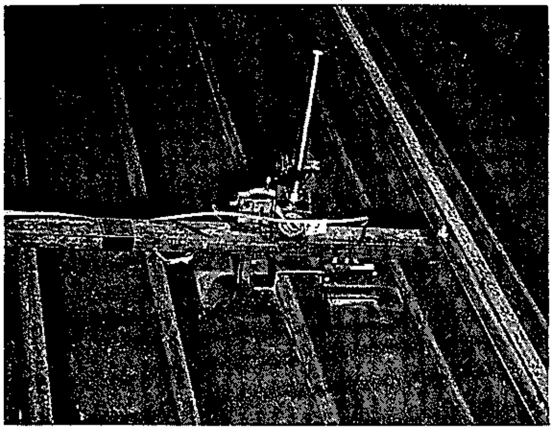

Figure 15

A PAIR OF POTENTIOMETERS MOUNTED TO MEASURE WEB MOVEMENTS

One pair was placed under each of the first two original load points (3 ft and 9 ft from centerline) and the third was placed midway between (6 ft from centerline). These "pots" had

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a range of 1.3 in. and were zeroed near midstroke. Dial indicators (Ames and Starret) with 1 in. ranges were also used during the first two tests to monitor transverse web movement at lower loads in order to check the performance of the "pots" and to provide immediate data for monitoring the tests (Figure 16). When the first test demonstrated the inability

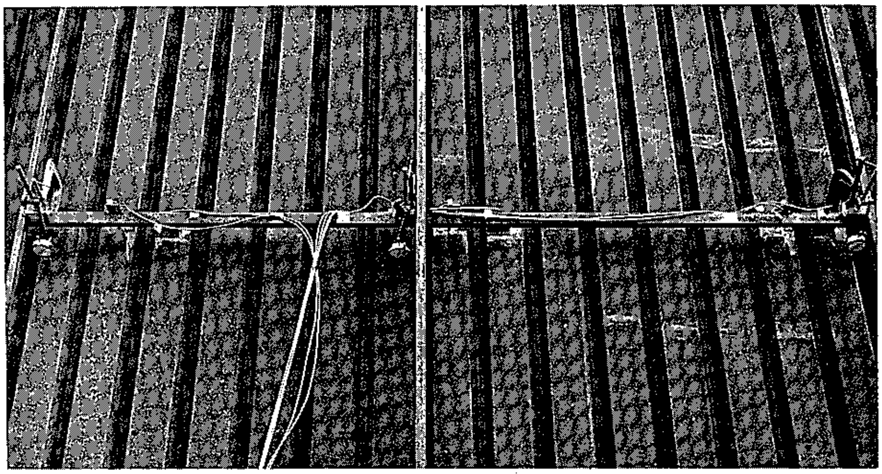

Figure 16

DIAL INDICATORS FOR MEASURING LATERAL WEB MOVEMENT

of this instrumentation ("pots" and dial gauges on web faces) to accommodate the relatively severe panel twisting and the undulating pattern of the web buckling, the three rib "pots" and outer two web "pots" were deleted. Their presence had been productive, however, through their earlier indication of the inadequacy of the original load distributing apparatus.

The basic instrumentation for the flange chord angles consisted of a (BLH SR4) strain gage for each chord angle mounted at the same distance from the sign centerline as the set of rosettes (18 in.) and the three sets of potentiometers (3 ft, 6 ft, and 9 ft). Since the original bearing blocks were 3 in. wide, the gages located at loading points were placed along the inside edge of the chord angle to clear the bearing blocks (Figure 7, Section III). To maintain consistency, all the other flange gages were similarly placed. This inside edge location proved fortuitous because of its ability to detect undesirable load concentrations at loading points.

Vertical deflections were measured by Lockheed WR8-15A position transducers. These instruments were linear potentiometers operated by a spring loaded flexible steel wire and had a 14-in. range. Two of these "wire pots" were attached 4 ft apart (laterally) at the loaded end to determine end rotation as well as deflection (Figure 6). A third "wire pot" was placed at the

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center of the other end. During the first test, noticeable deflection of the center of the support structure was observed and measured with a pocket tape at maximum load (approximately 1.1 in.). For the three subsequent tests, a fourth "wire pot" was utilized at this location to measure deflection throughout the loading sequence. The "wire pots" located at both ends of the sign structure and at the support column provided the necessary information to determine the true cantilever deflection of the sign structure. This procedure took into account both the deflection of the support structure and the rotation of the sign structure about its support.

The tensile load cells connected to the load transmitting strands to determine the magnitude of the applied loads were manufactured by the Materials and Research Department. They were threaded at the ends for attaching the strand gripping chucks. Their zero repeatability was stable, and their stress-strain function was linear throughout the load range of the testing apparatus.

To adequately monitor the effectiveness of the load distributing systems during the first test, additional strain gages were installed along the top inside edge of the top flange chord angles. Supplemental gages were also installed on the vertical legs of all the flange chord angles. Some of these determined the relationship of strains at those locations to the strains at the respective inside edge location; others indicated the mode and magnitude of stresses carried across the diaphragm along the lower chord angles. Two strain gages were attached to the tension side of the support post (the side facing away from the testing end) to detect any excessive strains on that member.

During the last three tests, the lower rosette on the unloaded side was monitored to detect any significant stresses that might be transmitted across the post diaphragms from the loaded side (none was indicated). A strain gage was mounted on the first web face panel inside the post diaphragm adjacent to the lowest rosette on the loaded side. It was oriented vertically in order to compare its readings with those of the vertical leg of the rosette and determine the relative severity of the web load on each side of the post diaphragm.

Data acquisition systems used for this project included a Digitec 50 channel system and a Hewlett Packard 25 channel system. Each instrumentation item used 1 channel except rosette gages, which used 3. Both systems produced digital data on printed tapes, but only the Digitec system was capable of utilizing accessory units to produce a punched tape in ASCII code for direct input into the ASR 33 teletype computer satellite at the Materials and Research Department laboratory. This satellite is part of a G.E. (General Electric) Time Sharing Service which utilizes a G. E. 235 computer.

The power source supplied 19.417 volts to the load cells and strain gages, and 200 millivolts to the potentiometers. Each strain gage was hooked up on the junction panel as a leg of a Wheatstone

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bridge (as shown in Exhibit 7 of the Appendix), and the bridge imbalance was measured and recorded by one of the data acquisition systems. The potentiometers were handled similarly (as shown in Exhibit 8 of the Appendix).

Zero and calibration readings were recorded with no loads applied to the structure. Zero readings were the reference values that were subtracted from later readings to measure net change. The calibration reading was measured with the test calibration resistor paralleled to the leg of the bridge adjacent to the leg containing the instrumentation circuit. The resulting bridge imbalance simulated a tensile strain of 1000 mu in. These readings were utilized to detect faulty circuits.

The computer service was also used to convert the indicated strains obtained from the rosette gages to actual strain components and then to convert the corrected strain components by a Mohr's circle analysis into maximum and minimum principal stresses, principal axis orientation, and maximum shear stress.

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V. TESTING OPERATIONS

The testing procedure followed was to increase the sign structure load by a 1.2 kip per jack load increment until the structure failed and, at each successive load level, record a set of instrumentation readings (a "run"). This loading increment was chosen so as to reach the designers' predicted failure load for the steel structure in ten equal intervals. The same increment was used for all project testing.

Zero and calibration runs were taken before and after each sequence of loading runs. Prior to each test sequence, at least one preliminary sequence up to the third or fourth load level was performed to check the loading and monitoring systems and to detect any irregular response by the sign structure. Zero and calibration runs were also performed before and after the loading distribution apparatus was installed on the sign structure in order to detect significant changes in the instrumentation zero readings caused by the added weight and to detect any damage to the instrumentation during the installation. Testing sequences and most preliminary sequences were performed early in the morning to minimize the strains induced by unequal thermal changes caused by exposure to the sun's rays.

The first loading sequences were performed in October and November 1968. These disclosed the necessity of utilizing a more elaborate loading structure. Redesigning, ordering the material for, and fabricating the members of the new system were completed by January 1, 1969. Due to administrative considerations, work was not resumed until the middle of March, and testing preparations were not completed until early April.

The first full test sequence was performed April 9, 1969, on the plug welded half of the steel sign structure. No significant indications of distress were evident until the load reached 1.8 kips per foot. Then vertical twisting of web face panels became visibly apparent, and the structure began emitting an occasional snapping or popping sound. The web twisting increased as more load was applied. At a load of 2.6 kips per foot, outward deflection of the vertical legs of the bottom chord angles was noticed above the support plate at the column. This deflection increased substantially as the load increased to 3.2 kips per foot. This was the maximum load applied to the structure since greater leg deflection might adversely affect the second half of the sign, which was yet to be tested.

When the load was withdrawn, a slight warp remained in the angles. The web material, at the connections to the twisted angles, sustained permanent deformation in the form of stress rings. These rings occurred at the first 7 connections out from the center line on both web faces and are shown in Figure 17.

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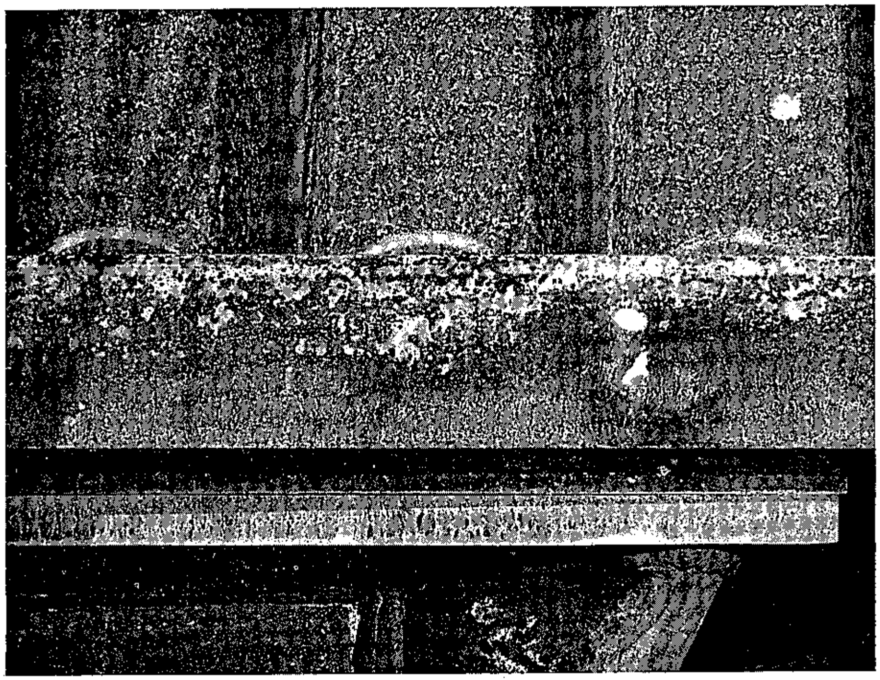

Figure 17

STRESS RINGS AT PLUG WELDS WHERE VERTICAL LEG OF CHORD ANGLE ROTATED OUTWARD ON STEEL STRUCTURE

The Huckbolted end of the steel structure was tested on April 29, 1969. As this structure was loaded, cracking and drumming sounds were produced and severe vertical twisting of the web ribs occurred. At a load of 2.2 kips per foot, the chord angles of the lower flange deflected outward slightly, and the side web on both sides of the diaphragm deflected inward. While approaching a load of 3.6 kips per foot, a sever undulating buckling of one side web suddenly occurred next to the post diaphragm, and the structure deflection increased substantially. The structure continued to support a substantial load, although less than at failure; but efforts to increase it only increased the deflection and longitudinal rotation of the structure (see Figure 21). The buckling pattern remained on one side when the load was removed and consisted of a convex buckle between two nearly vertical parallel concave buckles (Figure 18) while the other side sustained negligible permanent deformation. Each of the three buckles had a maximum deflection of 5 in. At the point where the trough nearest centerline intersected the top flange, the web material was torn about one Huckbolt fastener.

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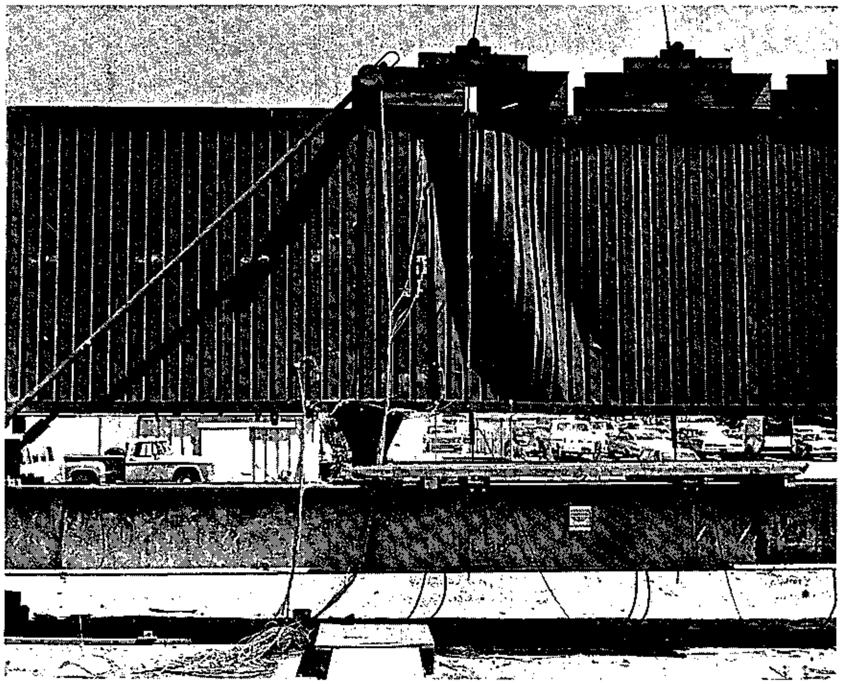

Figure 18

WEB FAILURE IN HUCKBOLTED END OF STEEL STRUCTURE

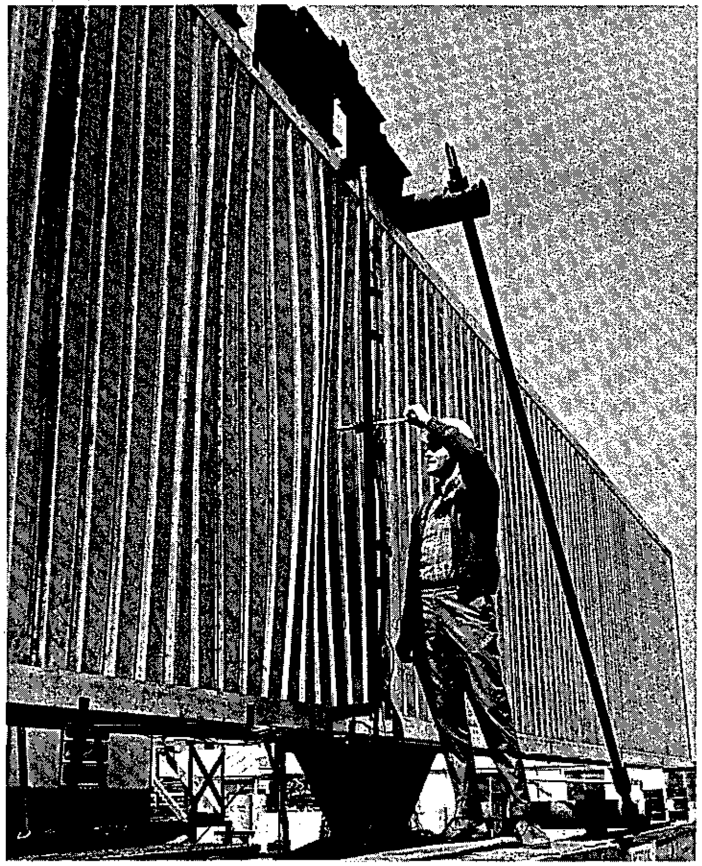

Figure 19

WEB DEFORMATIONS OF PLUG WELDED END ALUMINUM STRUCTURE MAXIMUM LOAD PRIOR TO FAILURE

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The plug welded portion of the aluminum structure was tested on May 13, 1969. Snapping and popping occurred more frequently as the load was increased. The web material on both sides of the post diaphragm bent inward and became more pronounced as loading increased. Vertical ripples in the web material were noticeable at a load of 0.8 kips per foot. The ripples grew to large slanted buckles which are shown in Figure 19. At a load of 1.7 kips per foot, a vertical welded web seam, located 4 ft from the centerline on one side of the sign structure, failed by cracking open 6 in. down the seam from the top of the web. The load was then released from the sign structure, which then appeared to sustain little permanent deformation. To reproduce the large buckles so that they might be photographed, the sign was again loaded to the same load level (1.7 kips per foot). The buckles regained their full depth of 2 in. (Figure 19); the tear in the web weld seam lengthened 2 in.; and a second seam 7 ft from centerline opened 4 in. down the seam at the top of the web. While holding a load of 1.7 kips per foot, the seam 4 ft from centerline failed completely, All of the plug welds connecting the top of the web to the upper chord angles failed between the failed vertical seam and a point 12 ft from centerline, and the welds connecting the bottom of the web to the lower chord angles failed from the failed vertical seam to a point 1.5 ft from the center. Intermittent weld separations occurred along the bottom flange from 4 ft to 11.5 ft from centerline. The sign deflected downward and twisted about its longitudinal axis. The sign, as it appeared after this failure, is shown in Figure 20.

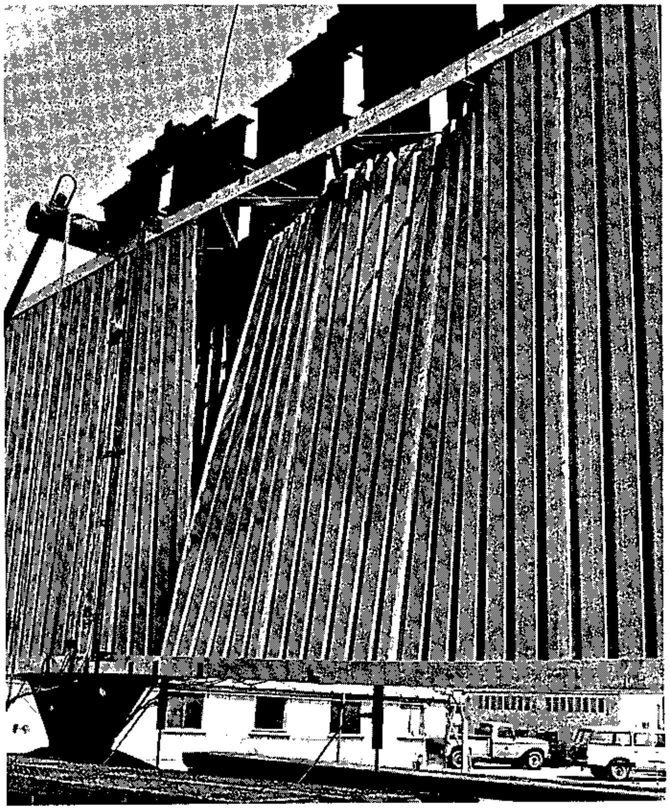

Figure 20

WEB FASTENER AND SEAM FAILURE OF PLUG WELDED END OF ALUMINUM STRUCTURE

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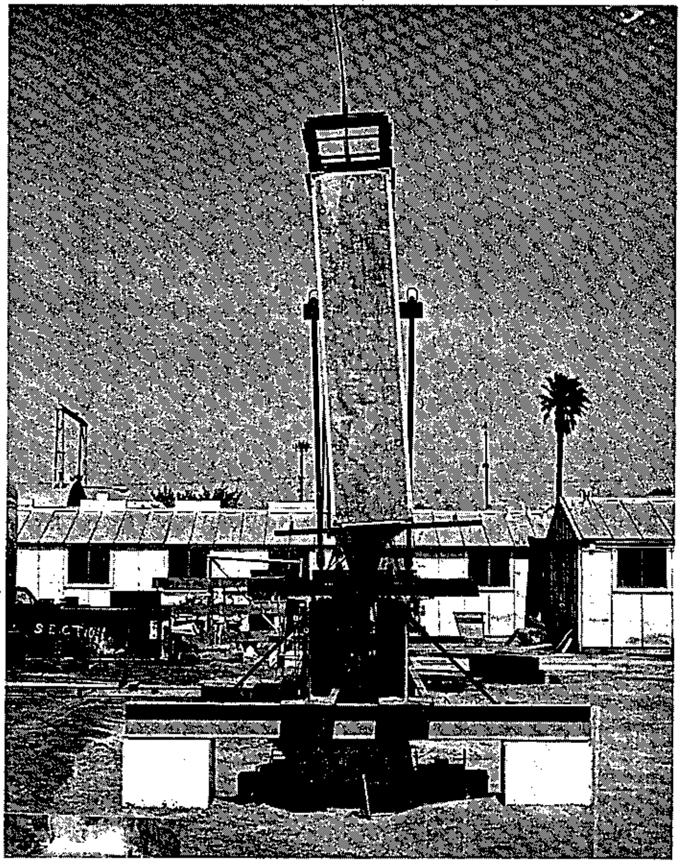

Figure 21

HUCKBOLTED END
OF ALUMINUM
STRUCTURE
AFTER TESTING

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The Huckbolted half of the aluminum sign structure was tested on May 23, 1969. The sign was loaded to 1.2 kips per foot before any significant indications of distress were evident. Ripples then appeared in the web and grew into large buckles. At a load of 1.65 kips per foot, the load began to drop and the sign structure continued deflecting downward. The failure occurred quietly and gradually. The sign was then unloaded to determine the structure's ability to regain its original shape. One web sustained permanent deformation in the form of shallow buckles. The load was again applied, but only a magnitude of 1.35 kips per foot could be attained. The buckles on the failed side deepened, and the sign twisted further about its longitudinal axis as shown in Figure 21. The buckles consisted of two convex and two concave troughs as shown in Figure 22. The two major buckles in the middle were 6 in. deep and the other two were 3 in. deep, measured from a reference plane at the face of the chord angles.

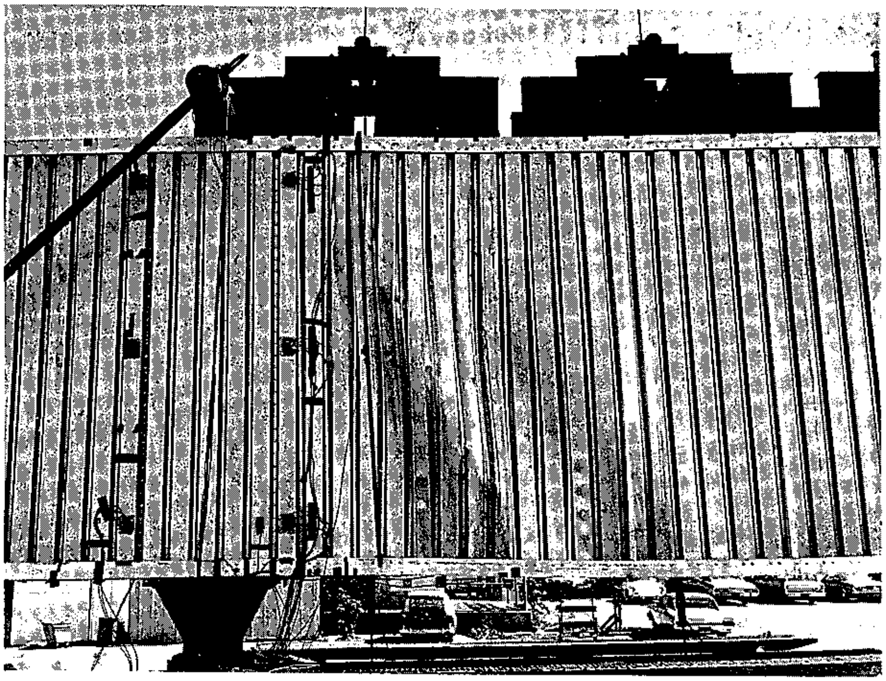

Figure 22

FAILED SIDE OF HUCKBOLTED END OF ALUMINUM STRUCTURE

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VI. SUMMARY

The loading apparatus satisfactorily accomplished its purpose of simulating a uniformly distributed load of sufficient magnitude to fail the sign. The instrumentation also functioned satisfactorily in acquiring the desired data which has been transmitted to the Bridge Department for evaluation.

The test results are summarized in the following table:

Sign Structure	Web to Flange Connection	Ultimate* Load	Mode of Failure
Steel	Plug Weld	3.11 kips/ft**	Flange chord angle deflection at support post and web distortion at connection to flange angles
Steel	Huckbolt	3.35 kips/ft	Severe web buckling and flange angle distortion near support
Aluminum	Plug Weld	1.57 kips/ft	Web connections and web seams
Aluminum	Huckbolt	1.59 kips/ft	Severe web buckling

* This is the maximum load level recorded. Actual failure occurred while the load was being increased to the next level. Estimates of the actual failure load appear in Section V.

** This test was terminated without a decisive failure in order to preserve the structural integrity of the other end of the structure which had not then been tested.

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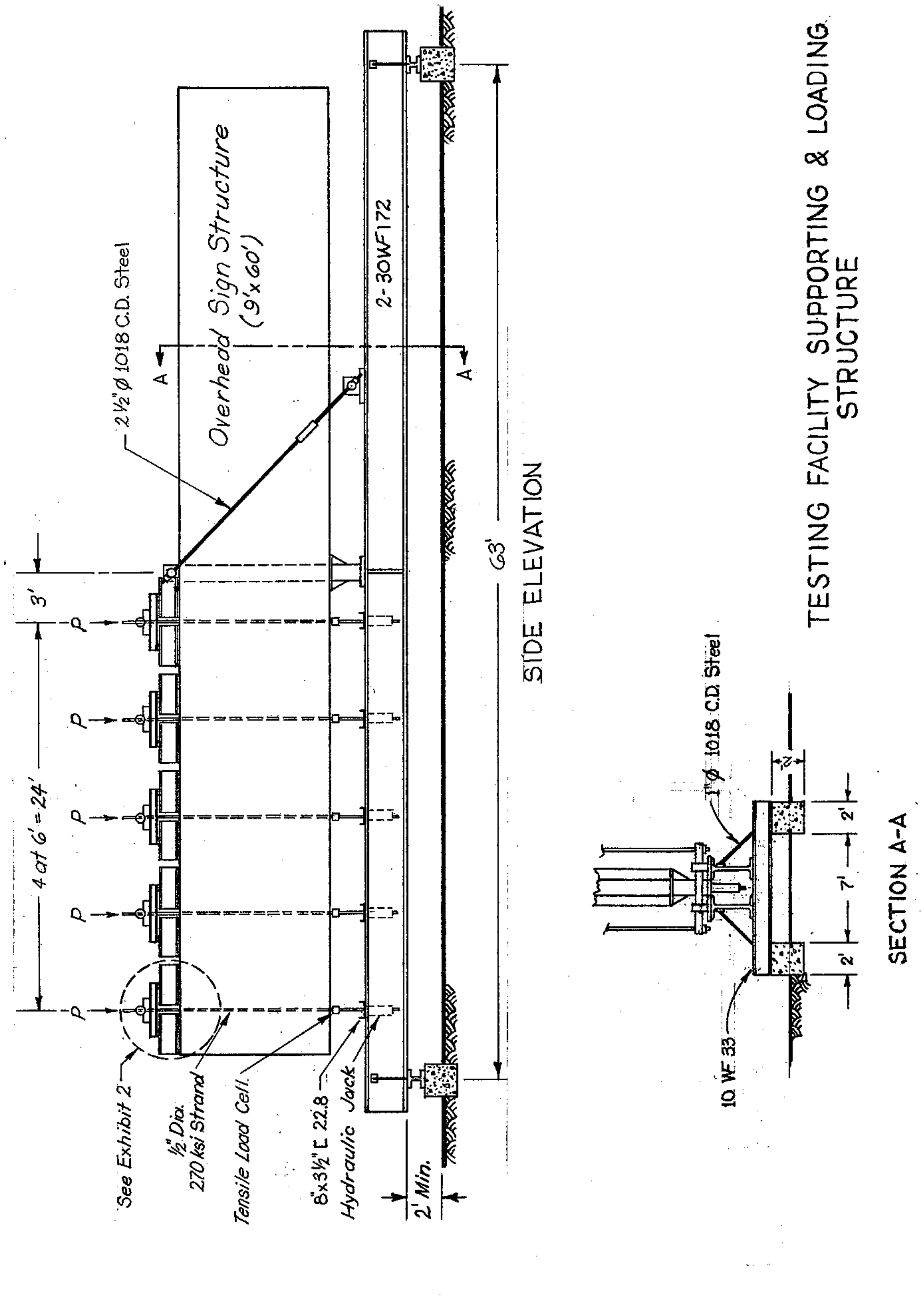
EXHIBIT 1



TESTING FACILITY SUPPORTING & LOADING STRUCTURE

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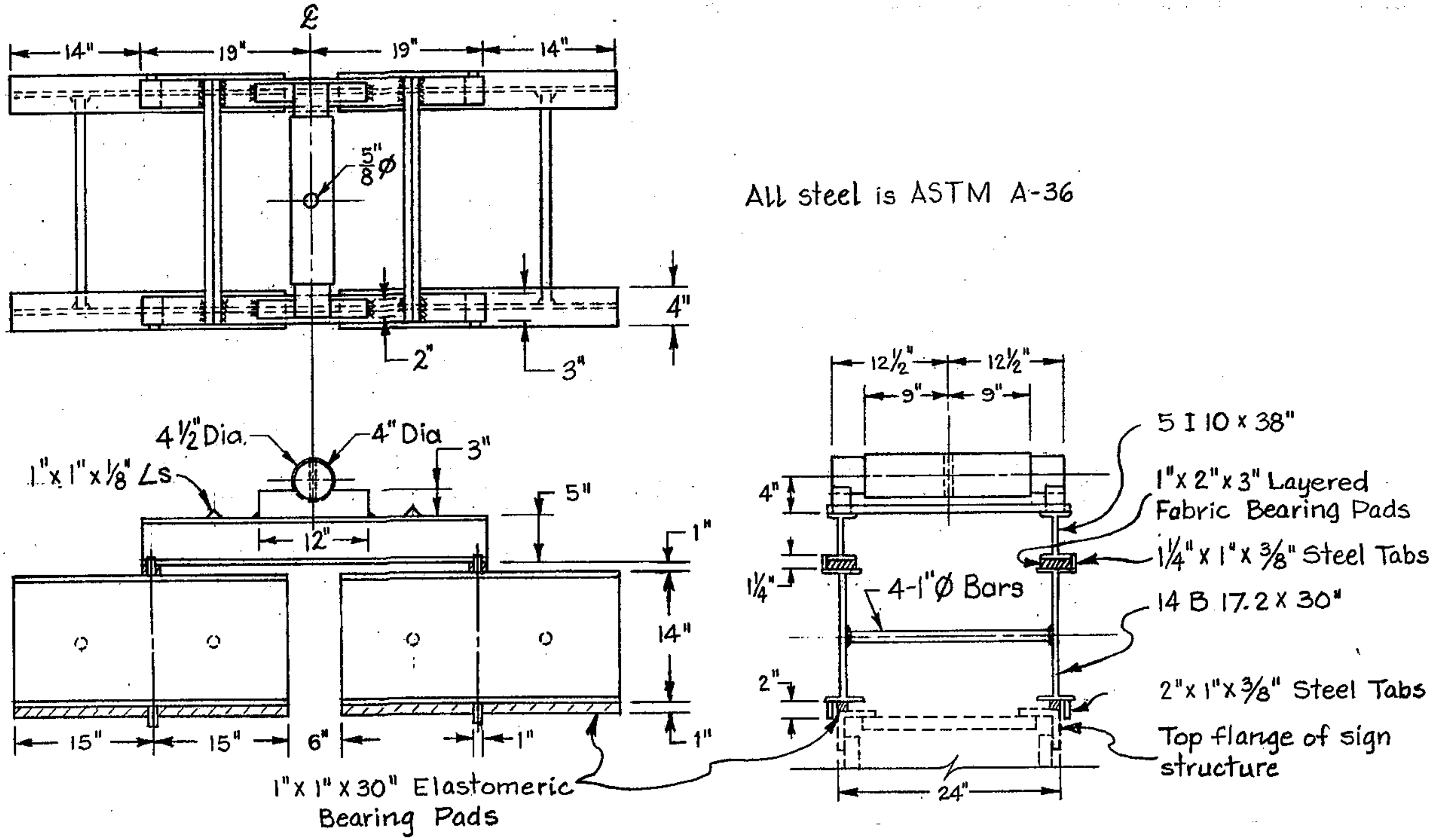
EXHIBIT 2



All steel is ASTM A-36

FINAL LOAD DISTRIBUTION APPARATUS

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EXHIBIT 3

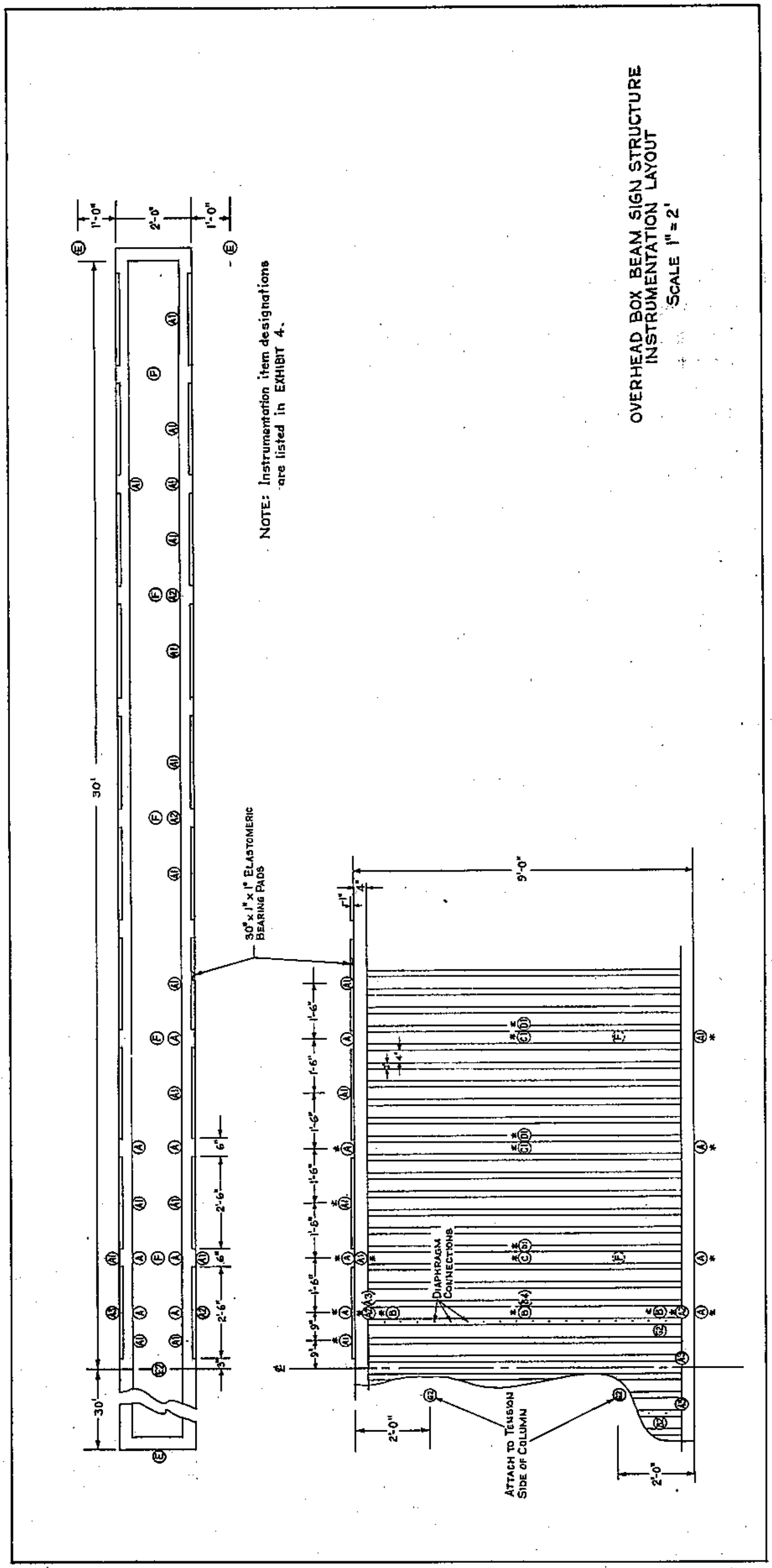

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EXHIBIT 4

INSTRUMENTATION DESIGNATIONS

Each instrumentation item is designated by a letter-numeral combination on Exhibit 3. Letters designate type and purpose. Numerals designate the tests for which each item was used. An asterisk indicates that there is an equivalent item located symmetrically on the structure. When each item of the pair has a different designation, that of the item on the back (north) side is shown in parenthesis.

Letter Designations

A = Strain gages for longitudinal flange strain

B = Rosettes for side web strain

C = Linear potentiometers for transverse web movement

D = Linear potentiometers for web rib distortion

E = Linear potentiometers for vertical deflections

F = Tensile load cells

G = Strain gages for vertical strain

Numeral Designations

No numeral = All four tests

1 = First test only

2 = Second, third, and fourth tests

3 = Second test only

4 = First, third, and fourth tests

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EXHIBIT 5

DATA FLOW CHART

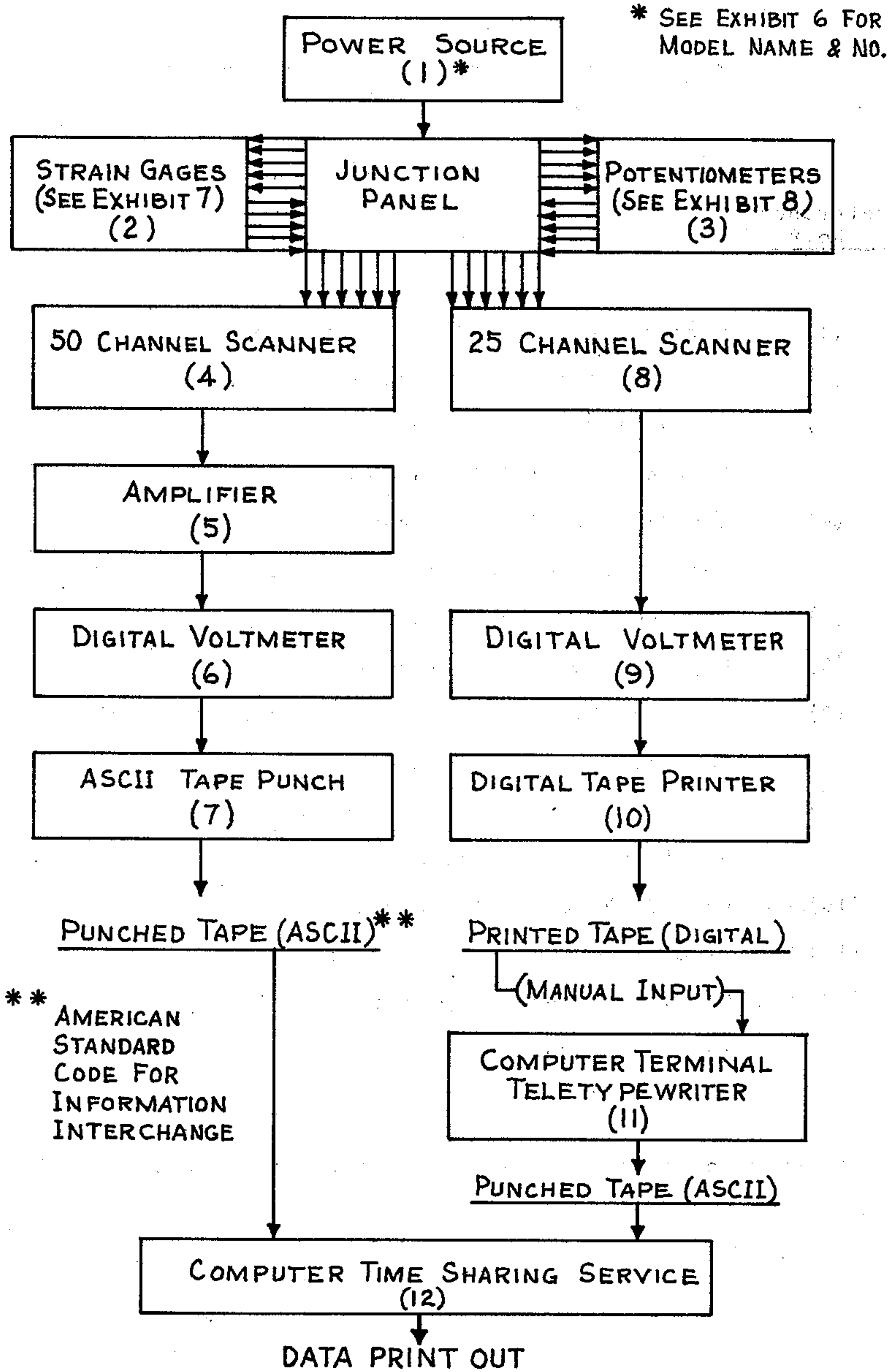

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EXHIBIT 6

DATA ACQUISITION AND PROCESSING EQUIPMENT LIST

Reference No.*	
(1)	Harrison 629-2A D.C. power source
(2)	BLH SR4 strain gages, rosettes, and load cells
(3)	Bourns 108 linear potentiometers
	Lockheed WRB-15A linear potentiometers
(4)	One Digitec 631 ten channel master scanner
	Two Digitec 633 twenty channel slave scanners
(5)	HP (Hewlett Packard) 2470A amplifier
(6)	Digitec 252-1 digital voltmeter
(7)	Digitec 671 tape punch and Digitec 623 punch controller
(8)	HP 2901A twenty-five channel scanner
(9)	HP 2401C digital voltmeter
(10)	HP S58562A digital recorder
(11)	Teletype ASR 33 teletypewriter
(12)	G. E. Computer Time Sharing Service (G. E. 235 Computer)

* Reference numbers correspond to item numbers in Exhibit 5.

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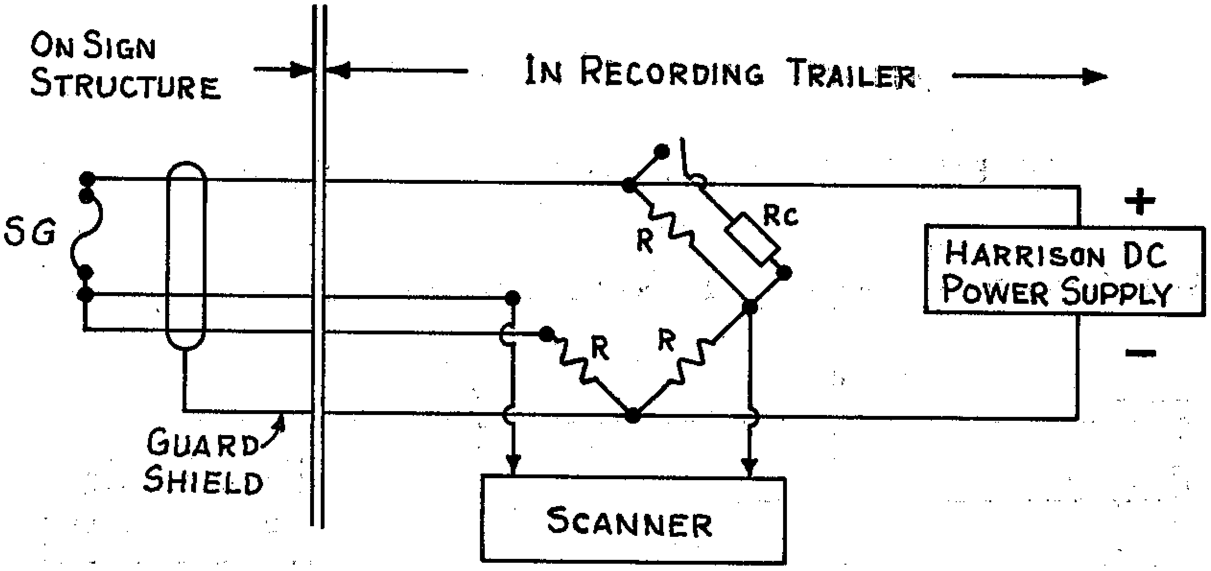
EXHIBIT 7

STRAIN GAGE CONNECTION DIAGRAM



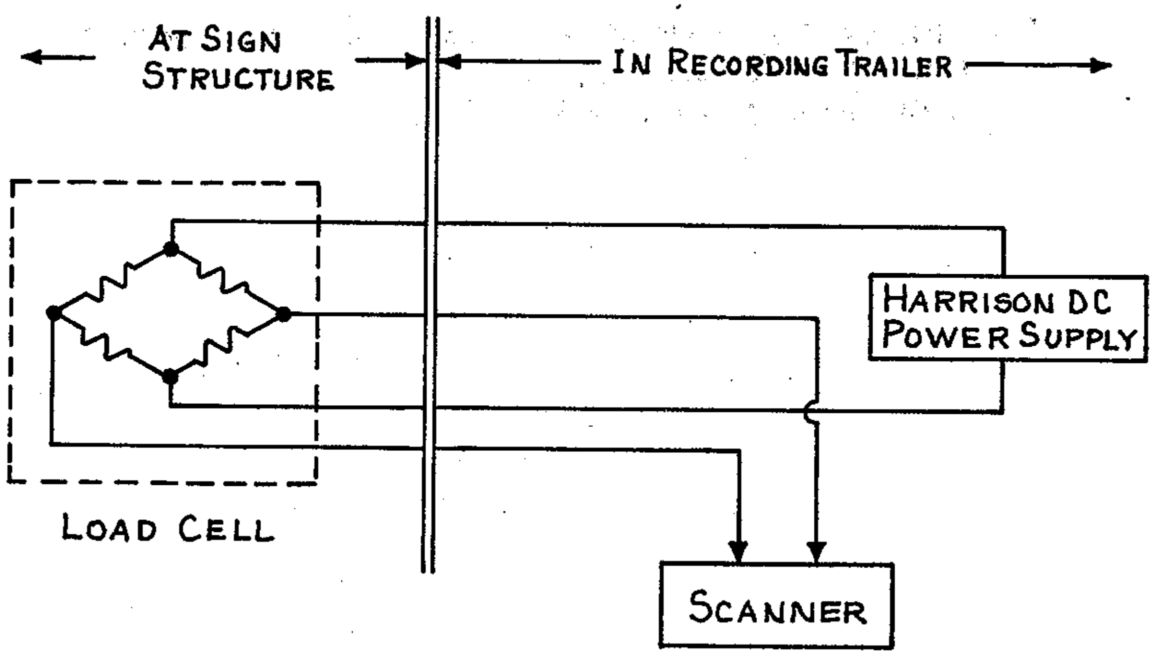
R = 350 ± 0.1% PERCISION RESISTOR

Rc = 169.1 KΩ SIMULATES 1000 INCH TENSION STRAIN.

SG = 350 OHM FAB - 0-35 STRAIN GAGE

D.C. VOLTAGE WAS SET AT 19.417 VOLTS TO GIVE A MICROINCH TO MILLIVOLT RATIO OF 10.

LOAD CELL CONNECTION DIAGRAM



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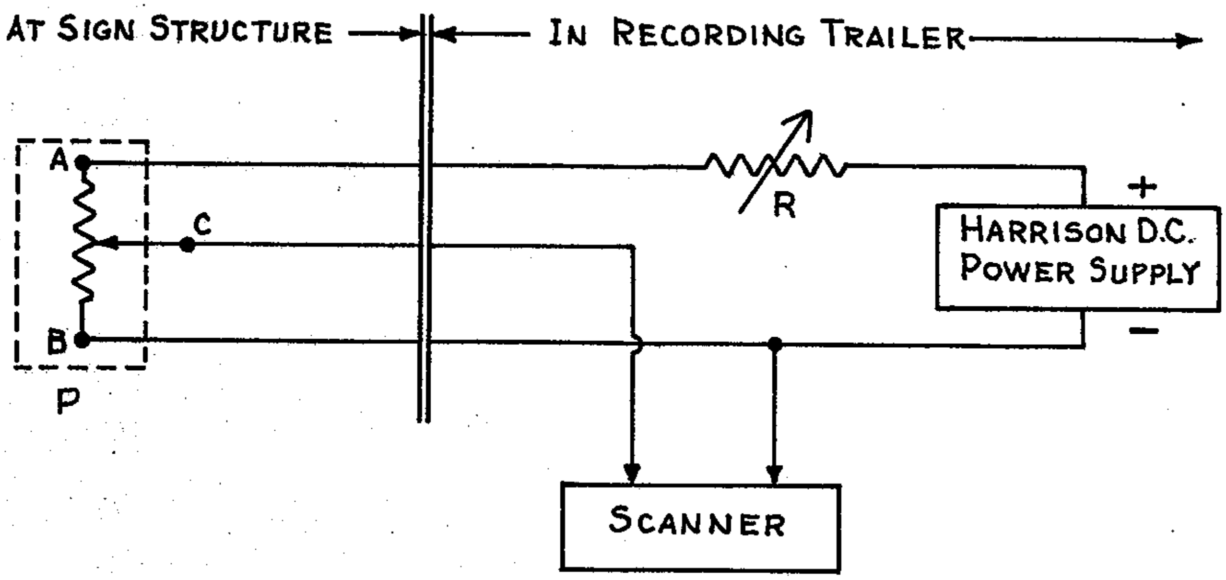
POTENTIOMETER CONNECTION DIAGRAM



R = VARIABLE RESISTOR SET AT APPROXIMATELY
14 K TO OBTAIN 200 MV AT TERMINALS A-B

P = LINEAR POTENTIOMETER POSITION TRANDUCERS
BOURNS MODEL 108, AND
LOCKHEED MODEL WR8 15A

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COMPUTER PROGRAM, TESTS 3 & 4

```
1FILES DIG10;DIG11
10:'1= ##.###  '2= ##.###  '3= ##.###  '4= ##.###  '5= ##.###
11:GAGE   STRAIN    LOAD  NO LOAD      GAGE   STRAIN    LOAD  NO LOAD
12: NO. (MU IN/IN)  (MV)    (MV)        NO. (MU IN/IN)  (MV)    (MV)
13:###    #####    #####  #####         ###    #####    #####   #####
14:ROSETTE   PRINCIPAL STRESS (PSI)  ANGLE   MAX. SHEAR
15:  GAGES           MAX.    MIN.      (DEG.)    (PSI)
16:### -###       ######  ######       ####     #####
17:LINEAR   LOAD  NO LOAD  DEFLECTION
18:POT.NO.   (MV)   (MV)   (MILLI-INCH)
19: ###     #####  #####     ###
110DIMI(50),D(50)
160FORJ=1TO50
170READ#1,B,I(J)
180NEXTJ
190LETC=1
230READD$,N$
233IFD$="END"THEN1530
240FORJ=1TO50
245READ#C,B,D(J)
250IFEND#CTHEN270
260GOTO280
270LETC=C+1
280NEXTJ
340LETP=50
360IFN$="INITIAL CALIB."THEN390
370IFN$="POST CALIB."THEN390
380GOTO400
390LETP=22
400FORL=1TO15
450PRINT
460NEXTL
470PRINTTAB(20);"SIGN STRUCTURE TEST"
480PRINTTAB(12);"ALUMINUM, FIRST HALF (PUDDLE WELD)"
490PRINT
500PRINT
510PRINT"DATE ";D$,"RUN NO. ";N$
520PRINT
530PRINT"DUMMY (GAGE #38)= ";(D(38)-I(38))*10;" MU IN"
535PRINT"VOLTAGE (GAGE #50)= ";D(50)/10;" MV"
536PRINT
540PRINT"JACK LOAD IN KIPS"
541LETB1=(D(41)-I(41))*.05759
542LETB2=(D(42)-I(42))*.05821
543LETB3=(D(43)-I(43))*.05795
544LETB4=(D(44)-I(44))*.05795
545LETB5=(D(45)-I(45))*.05770
550PRINTUSING10,"#",B1,"#",B2,"#",B3,"#",B4,"#",B5
580 PRINT
590PRINTUSING11
600PRINTUSING12
610 PRINT
620FORJ=1TO18
640LETB1=(D(J)-I(J))*10
645LETB2=(D(J+19)-I(J+19))*10
660PRINTUSING13,J,B1,D(J),I(J),J+19,B2,D(J+19),I(J+19)
667NEXTJ
670LETJ=19
680PRINTUSING13,J,(D(J)-I(J))*10,D(J),I(J)
700IFP=22THEN230
710PRINT
```

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COMPUTER PROGRAM, TESTS 3 & 4

```
740PRINTUSING14
750PRINTUSING15
760PRINT
770LETG=T=S1=S2=A=0
780FORJ=20TO37STEP3
790LETR1=(D(J)-I(J))*10
800LETR2=(D(J+1)-I(J+1))*10
810LETR3=(D(J+2)-I(J+2))*10
820LETR1=R1-R3/200
830LETR2=1.02*R2-(R1+R3)/200
840LETR3=R3-R1/200
850LETG=SQR(ABS((R1-R3)↑2+(2*R2-R1-R3)↑2))
860LETS1=5*((R1+R3)/.7+.769*G)
870LETS2=5*((R1+R3)/.7-.769*G)
880LETT=3.85*G
890LETT1=(2*R2-R1-R3)/((R1-R3)+.0001)
900IFT1<0THEN940
910IFT1>0THEN980
920LETA=0
930GOTO991
940LETT1=-1*T1
950LETA=ATN(T1)
960LETA=-1*(A/2)
970GOTO991
980LETA=ATN(T1)
990LETA=A/2
991IFR1>=R3THEN1000
992IFA<=0THEN995
993LETA=A-(3.14159/2)
994GOTO1000
995LETA=A+(3.14159/2)
1000PRINTUSING16,J,J+2,S1,S2,A*57.3,T
1020NEXTJ
1025PRINT"(ASSUMPTIONS: YM=10 MIL, MU=0.3)"
1030PRINT
1120PRINTUSING17
1130PRINTUSING18
1140PRINT
1150FORJ=39TO40
1155LETB7=(D(J)-I(J))/1.45
1160PRINTUSING19,J,D(J),I(J),B7
1170NEXTJ
1175PRINT
1180FORJ=46TO49
1185LETB8=(D(J)-I(J))*8
1190PRINTUSING19,J,D(J),I(J),B8
1191NEXTJ
1193LETB9=((D(46)-I(46))/2+(D(47)-I(47))/2-(D(48)-I(48))*2+(D(49)-I(49))
1194PRINT
1195PRINT"NET DEFLECTION AT LOADED END= ";B9;" MILLI-INCHES"
1200IFN$<>"POST ZERO"THEN230
1210FORJ=1TO50
1220LETI(J)=D(J)
1230NEXTJ
1240GOTO230
1520DATAEND,0,DATE
1530FORL=1TO15
1540PRINT
1550NEXTL
1560END
```

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EXHIBIT 10

TYPICAL PRINTOUT, TESTS 3 & 4

SIGN STRUCTURE TEST
ALUMINUM, SECOND HALF (HUCKBOLT)

DATE 5-23-69 RUN NO. 8

DUMMY (GAGE #38)= 990 MU IN
VOLTAGE (GAGE #50)= 199.6 MV

JACK LOAD IN KIPS
#1= 9.733 #2= 9.546 #3= 9.562 #4= 9.388 #5= 9.521

GAGE NO.	STRAIN (MU IN/IN)	LOAD (MV)	NO LOAD (MV)	GAGE NO.	STRAIN (MU IN/IN)	LOAD (MV)	NO LOAD (MV)
1	1810	540	359	20	200	98	78
2	640	264	200	21	-560	405	461
3	620	571	509	22	-260	305	331
4	730	1218	1145	23	1160	859	743
5	350	69	34	24	300	796	766
6	130	72	59	25	-200	351	371
7	1790	591	412	26	560	259	203
8	1330	324	191	27	-240	229	253
9	1190	61	-58	28	-640	102	166
10	620	119	57	29	280	828	800
11	-90	-23	-14	30	-490	279	328
12	-2090	50	259	31	-370	599	636
13	-1130	254	367	32	650	356	291
14	-440	14	58	33	120	198	186
15	-370	229	266	34	-170	22	39
16	-2090	-93	116	35	350	15	-20
17	-730	109	182	36	-120	-74	-62
18	-1030	33	136	37	-350	27	62
19	-510	56	107				

ROSETTE GAGES	PRINCIPAL STRESS (PSI) MAX.	MIN.	ANGLE (DEG.)	MAX. SHEAR (PSI)
20 - 22	4098	-4950	-33	4530
23 - 25	12250	1395	-7	5434
26 - 28	4328	-5465	-9	4903
29 - 31	3664	-4944	-27	4310
32 - 34	6709	113	-8	3302
35 - 37	2864	-2864	-9	2867

(ASSUMPTIONS: YM=10 MIL, MU=0.3)

LINEAR POT.NO.	LOAD (MV)	NO LOAD (MV)	DEFLECTION (MILLI-INCH)
39	1963	1317	445
40	1959	1144	562
46	1506	1939	-3464
47	1449	1865	-3328
48	1705	1773	-544
49	414	311	824

NET DEFLECTION AT LOADED END= -1484 MILLI-INCHES

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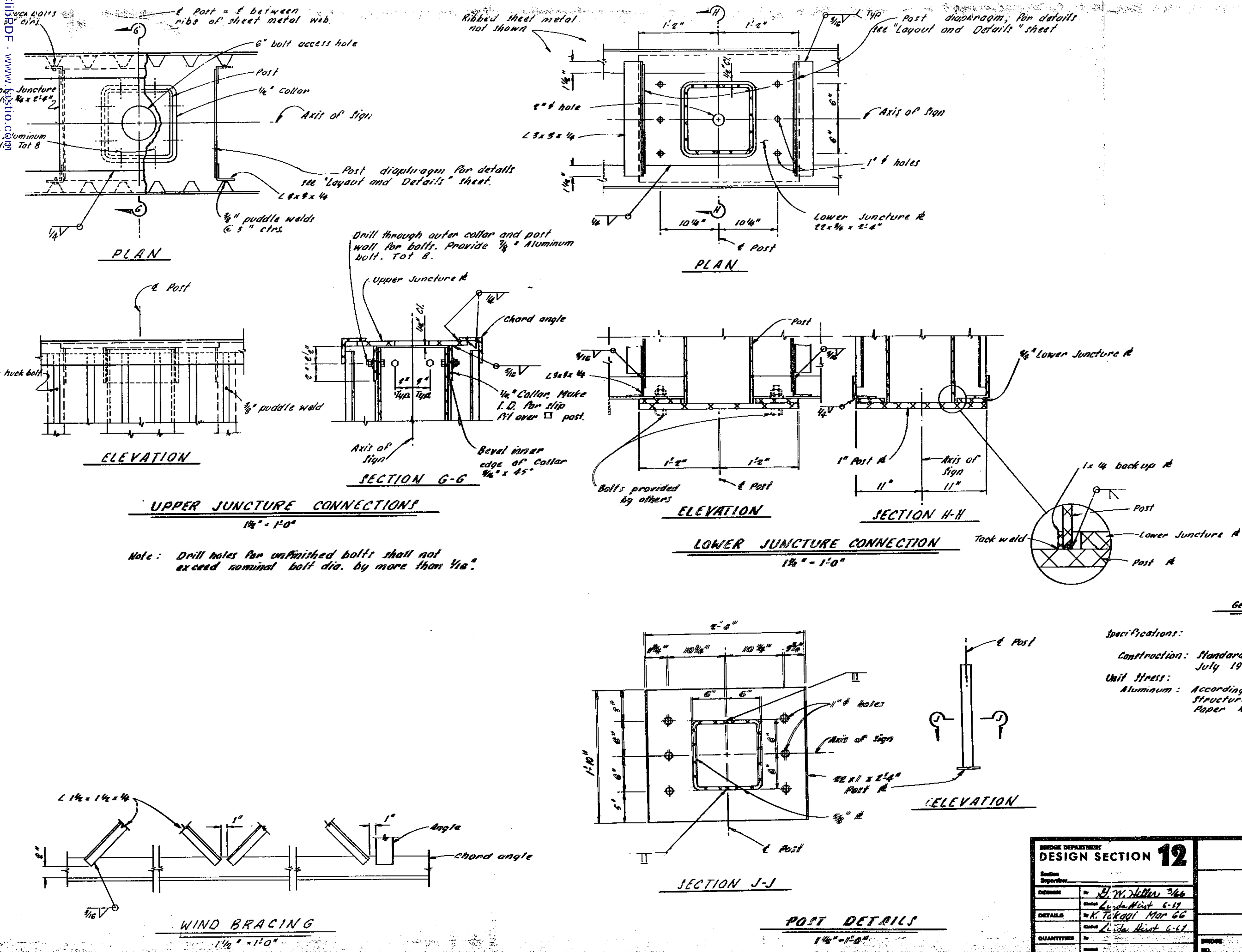
Fig. #11



Note: Drill holes for unfinished bolts shall not exceed nominal bolt dia. by more than 1/16".

General Notes

Specifications:

Construction: Standard Specifications, Divison of Highways, dated July 1964 and the Special Provisions.

Unit Stress:
Aluminum: According to A.S.C.E. "Suggested Specifications for Structures of Aluminum Alloys" ASCE Proc. Paper N° 3341 dated December 1962.

BRIDGE DEPARTMENT
DESIGN SECTION 12

Design: G. W. Heller 3/66
Details: K. Takagi Mar 66
Checked: Linda Hirst 6-67

STATE OF CALIFORNIA
HIGHWAY TRANSPORTATION AGENCY
DEPARTMENT OF PUBLIC WORKS
DIVISION OF HIGHWAYS

ALUMINUM BOX BEAM SIGN
FOR TEST

DETAILS NO. 2

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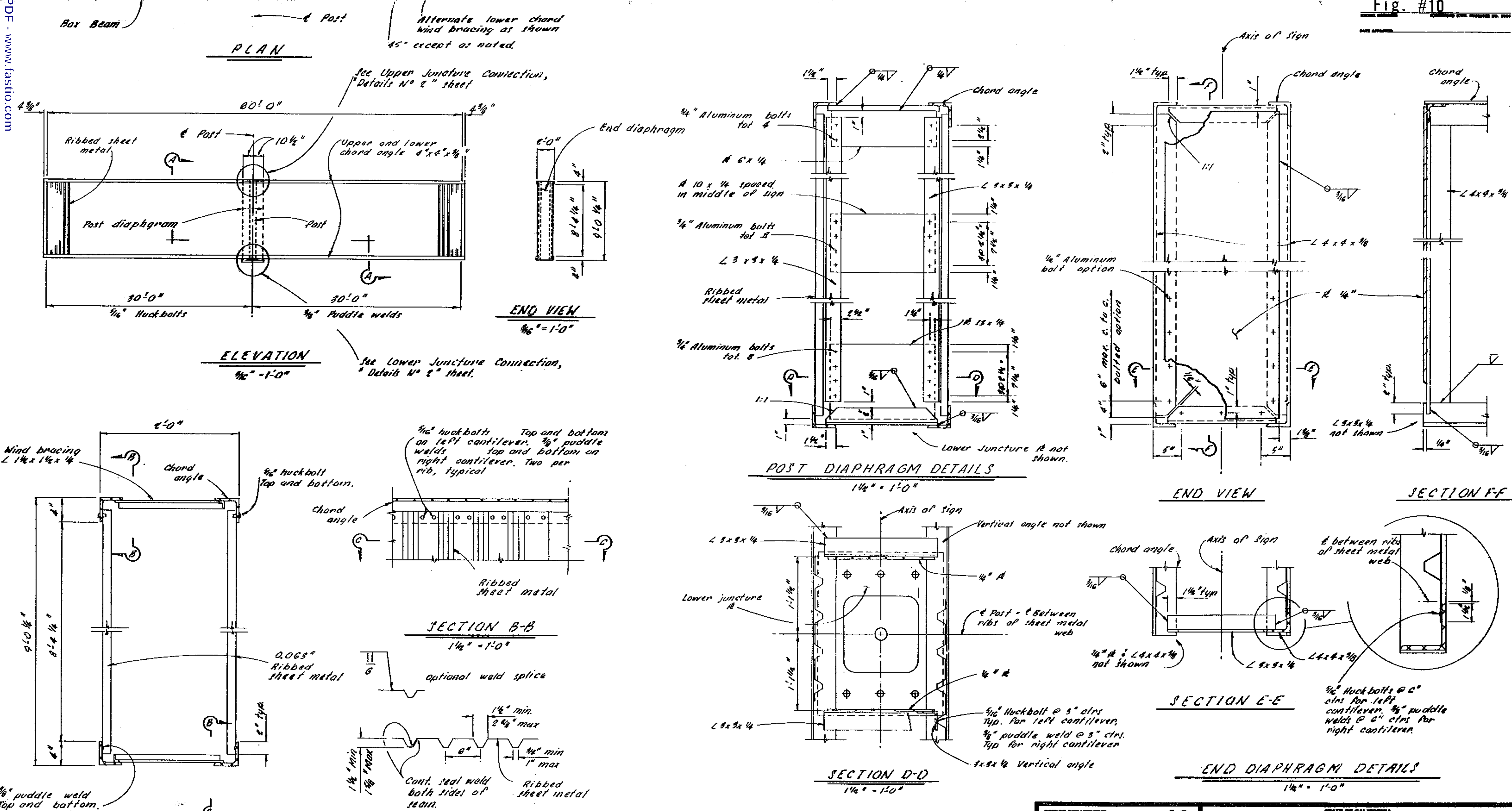
Fig. #10



INDEX TO PLANS

Sheet Nº	Title
1.	Layout and Details
2.	Details Nº 2

BRIDGE DEPARTMENT
DESIGN SECTION 12

DESIGN: G. W. Keller 3/66; Linda Hirst 6-67
DETAILS: K. Takagi Mar 66; Linda Hirst 6-67

STATE OF CALIFORNIA
HIGHWAY TRANSPORTATION AGENCY
DEPARTMENT OF PUBLIC WORKS
DIVISION OF HIGHWAYS

ALUMINUM BOX BEAM SIGN
FOR TEST
DETAILS NO. 1

WO
CU

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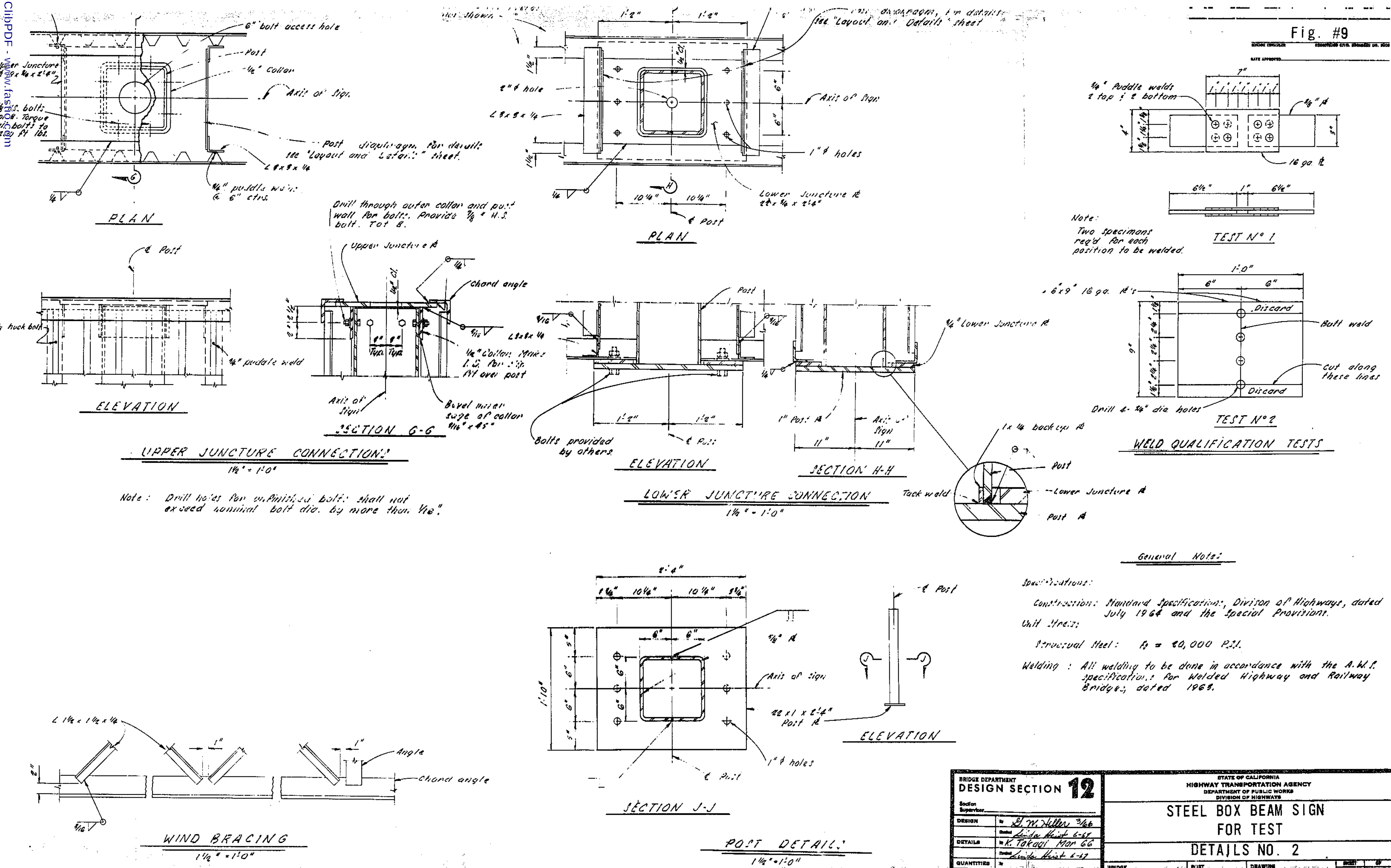
Fig. #9



General Notes:

Specifications:

Construction: Standard Specifications, Division of Highways, dated July 1964 and the Special Provisions.

Unit Stress:

Structural Steel: f_s = 20,000 P.S.I.

Welding: All welding to be done in accordance with the A.W.S. specifications for Welded Highway and Railway Bridges, dated 1963.

BRIDGE DEPARTMENT
DESIGN SECTION 12

DESIGN	G. W. Hiller 3/66
Checked	Linda Hirst 6-67
DETAILS	K. Takagi Mar 66
Checked	Linda Hirst 6-67
QUANTITIES	

STATE OF CALIFORNIA
HIGHWAY TRANSPORTATION AGENCY
DEPARTMENT OF PUBLIC WORKS
DIVISION OF HIGHWAYS

STEEL BOX BEAM SIGN FOR TEST

DETAILS NO. 2

BRIDGE NO. | POST MILE | DRAWING NO.

Discard prints bearing earlier revision dates

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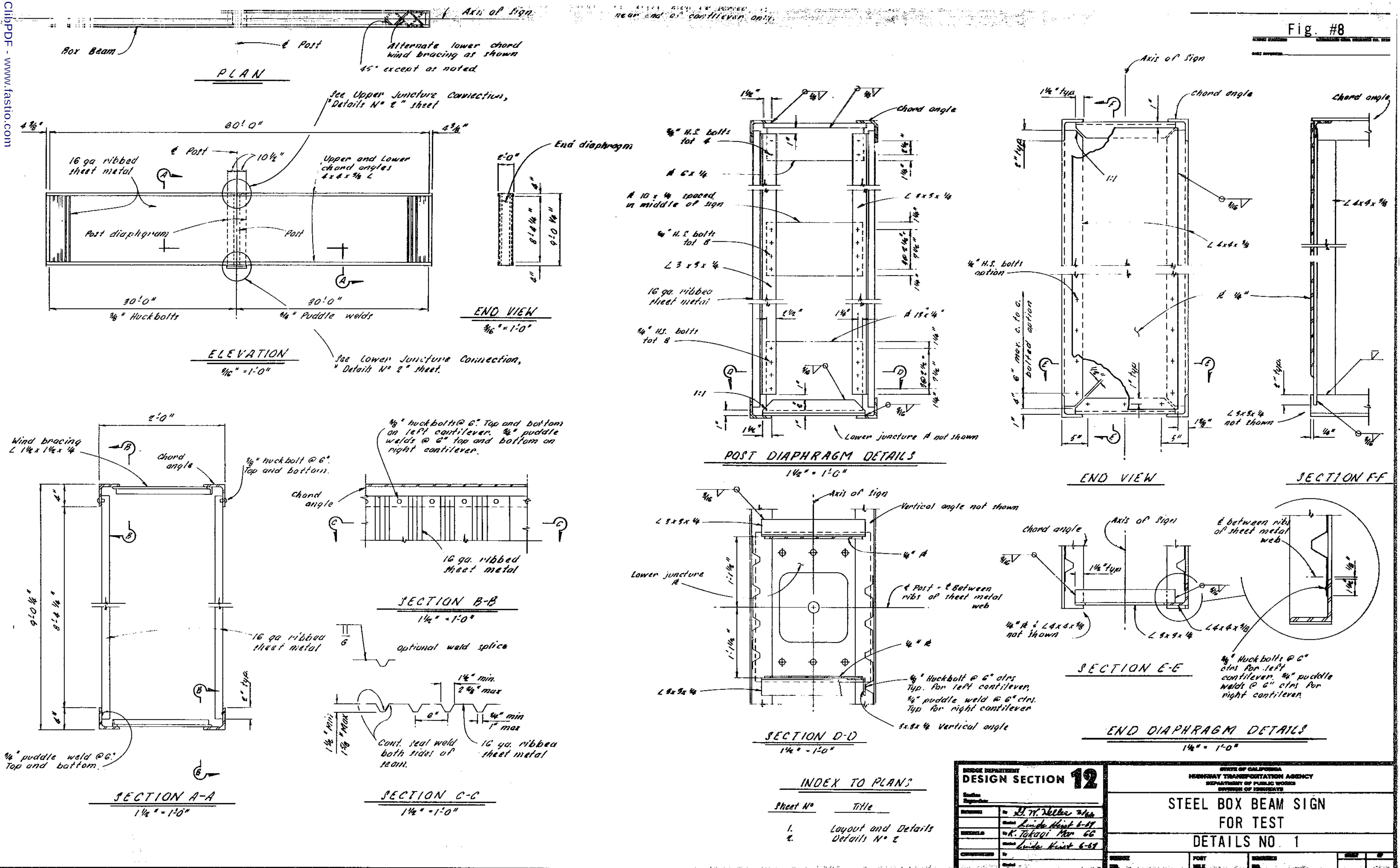
Fig. #8



INDEX TO PLANS

Sheet N°	Title
1.	Layout and Details
2.	Details N° 2

BRIDGE DEPARTMENT
DESIGN SECTION 12

STATE OF CALIFORNIA
HIGHWAY TRANSPORTATION AGENCY
DEPARTMENT OF PUBLIC WORKS
DIVISION OF HIGHWAYS

STEEL BOX BEAM SIGN
FOR TEST
DETAILS NO. 1

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